Engineering Record

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Co-operation and Effectiveness

THE unusually intelligent and close co-operation among the engineering, operating and purchasing departments and the broad-gage policy of the executive officers are the dominant factors in the success of the track-elevation department of the Rock Island Railroad, as outlined in this issue. Had the engineer in charge been hampered by red tape the sliding-wage scale would have been impossible. A spirit of good-natured rivalry exists among different gangs on the same operations and has tended to speed up the work. Before construction was started the engineer of track elevation, as well as a number of contractors, was asked to make a bid on the work. His was the lowest figure. The Rock Island, as a result, is doing the work with its own forces, and the effectiveness of the organization is shown by the fact that it is thousands of dollars below its own bid. Again, not a single train delay has occurred to date, although a 100-ft. right-of-way gives little room to work in.

Averting Foundation Disasters

LARGE and expensive five-story brick buildings located in a hollow, on the surface of a deep bed of soft muck, afford a combination that should impress even the layman with the necessity for the most competent and reliable foundation design and construction. The conspicuous absence of one or both of these requisites in the case described on page 125 resulted in a dangerous and disgraceful condition of the nearly completed structure and involved a heavy outlay to prevent a still greater loss by the imminent total destruction of the building. Although it is the duty of the architect to consult with engineer specialists on difficult and dangerous construction, this is too seldom done, and, therefore, the owner, even though possessing full confidence in his architect, should protect himself by assurance that such consultations have been held, or by himself retaining competent engineering advisers. The design having been properly adapted to the conditions, it remains to be seen that it is correctly executed. This duty is, naturally, in the architect's province, but in any event it should be specifically provided for. Contemplating the case under discussion the wonder is not that there has been danger of disaster in this particular case but that, with the absence of engineering advice, these cases do not occur oftener. The repair work in such a case is, of course, interesting. The use of steel piles has been often described, but it is evident that the supporting and adjusting of the superstructure, together with the determination of the amounts, character, and position of the new work to secure both economy and safety, involved careful judgment and long experience. Had services of equal grade been requisitioned for the original construction the owners of the buildings would have been large gainers, both by the reduced first cost and by the earlier receipt of rentals.

Making Reports Say Something

FROM frank interchange of engineering experience comes the gradual advancement of the arts associated with municipal problems. The New Orleans officials recognize this fact and allude to it in the preface of the latest report of the Sewage and Water Board. Semi-annual reports are issued. The mid-year report is of passing interest only and is not distributed generally. The year-end report is made as complete as possible, without unessential details, and is exchanged with other cities and sent to those associated with and interested in similar municipal problems with the view of adding whatever New Orleans may have of value to the growing fund of common knowledge. The Engineering Record has repeatedly urged the necessity of publishing reports containing matter of interest to the engineer. No city can be self-sufficient from the engineering standpoint. Summing up the year's work should include something more than the location of new fire hydrants and city-main extensions. Publishing this sort of information may be necessary to keep the city printer busy, but, as in New Orleans, the report for wide distribution should contain only meaty material with unessentials left out. Writing a yearly report is often left to the end of the year and then accomplished as an extra-rush job. As fast as history is made and interesting features of construction are completed they should be recorded. Then the annual report will not make its appearance a year late.

Engineering by British Architects

THE description in this journal, of July 18, of a British architect's plans for the preservation of ancient wooden roof trusses at a cost of about $300,000 for work which will last six years demonstrates the existence of another field for capable and ingenious structural engineering work. It is not, however, at all evident that engineering plans would be welcomed or accepted in this matter. In an editorial comment on the conservation of the Westminster Hall roof the London "Times" characterizes it as "an instance of the encroachment of the architect on the province of the engineer," but explicitly disclaims any reflection on the merits of the scheme. It is neither remarkable nor wrong that great pride should be felt in a structure of such antiquity and of great historical interest and that pains should be taken to preserve its integrity. Although the height and span of the roof may have been unprecedented and the structure a wonderful achievement when designed, its proportions, 65½ ft. span by about 60 ft. high above the wall seats, and probably less than 200 ft. long, are very ordinary for the modern constructor and it is probable that the lines of the original design together with the color and finish of the timber and interior effect could have been entirely preserved and safeguarded by the simple expedient of suspension from exterior trusses in a fraction of the time and at a fraction of the cost that is allowed for the elaborate patching of the old timber, its combination with steel trusses, and the use of massive supporting falsework instead of a light traveler platform for the workmen.

Clogged Sewage Filters

THE announcement of the proposed abandonment of the intermittent filters that have been in use for a number of years for purifying the sewage of New Britain, Conn., is likely to arouse considerable interest among engineers who are concerned with sewage treatment. Intermittent filtration skilfully conducted where suitable areas for practising it are available unquestionably gives better results for the same cost than any other method of treatment. Unfortunately, only very few cities have such areas, and therefore the amount of study given to intermittent filtration is comparatively little and the more intensive methods using settled sewage and coarse filters have a more prominent place in engineering discussions. The trouble at New Britain is stated to be a clogging of the sand, due mainly to the large quantity of pickling liquors discharged into the sewage from manufacturing plants. That this waste causes some trouble is not surprising in view of the experience with it at Worcester, Mass., but that it should necessitate the abandonment of intermittent filtration will be disappointing to those who believe that the natural local resources of a district should be utilized so long as it is economical to do so. Worcester's sewage is peculiarly difficult to treat on account of the pickling liquors and iron-works wastes in it, yet the filters of that city have been kept in service for many years, and it is only because no more sand areas are available that the city will have to construct works of other types for treating a part of its sewage. Where intermittent filtration on local sand areas is practicable it is usually economical, and it will be unfortunate for civic economics as well as for the extension of engineering knowledge if the reasons for the lack of success at New Britain and the success at Worcester are not studied. Expensive intensive treatment plants are more interesting engineering works than the less costly and less conspicuous filter beds, and there is a tendency to neglect the latter in spite of their utility when available and well managed.

Decision on Turbid Water

THE Public Service Commission of Nevada recently decided a case in which the turbidity of water supplied to Reno by the Reno Power, Light & Water Company was the mooted point. The decision is of interest to sanitary engineers because it fixes definite turbidity standards and deals, not with the question of the bacterial content of the water or whether or not it is injurious to health, but with its clarity and freedom from tastes and odors.

After weighing the expert testimony the water company was ordered to install a coagulating plant and a standard of clearness was prescribed by the terms of which the maximum turbidity of the treated water is fixed at 30 parts per million and the monthly average, based upon daily readings, is not to exceed 15 parts per million. The substance of the complaint by the city of Reno was that the defendant company was not furnishing water of good quality for domestic use. The answer of the defendant did not deny that the water was, at times, turbid, but it was alleged that the turbidity was caused by storms and conditions which it was impossible to control. The bad taste of the water was ascribed to the discharge into the Truckee River of refuse from a paper mill located upstream.

This, then, is a case in which the physical quality of a water is the simple element of controversy, and from the commission's ruling it is apparent that the character of a supply may be ordered to be improved even though the water may not be a source of direct danger to the health of a community.

In discussing the water company's contention that the cost of the proposed improvements would reduce its profits below 4½ per cent net, the commission has this to say: "A company furnishing a municipality with water for domestic use is not privileged to furnish water of poor quality which is, or may be, offensive to the taste, of bad odor, or deleterious to health, simply because its profits are small, or, in fact, though it may be conducting its business at an actual loss. Under any circumstances the company is bound to make every reasonable effort to improve the water. Therefore the value of the plant and the earnings of the company will not be considered in this case. The quality of the water and the duties of the company with respect to it will be regarded as the sole issues for determination."

Taken as a detached quotation and without knowledge of the state of development of the company's plant this is a very harsh doctrine. One would expect the commission to suggest that later a petition to increase the rates would be in order if a reduction below a 6 per cent return resulted from the installation of a filtration plant. A company in the development stage, of course, must expect to lose money for a time, but the Reno system apparently is beyond that stage.

Sewerage Statistics

ABOUT fifteen years ago engineers engaged in municipal work were devoting considerable attention to devising uniform methods of accounting in connection with the construction and operation of waterworks. One reason for this was that consumers in cities where high rates prevailed insisted that the rates should be lowered to the level of those in other cities, and in the absence of uniform systems of accounting it was impossible to present detailed figures to prove that the high rates were due to inevitably greater fixed and operating charges rather than to a desire to get all the revenue the consumers would yield without a fight to a finish. Another reason was that engineers had learned that cost accounting was highly important in showing how work was going, and that comparisons of costs for similar work in other years or in the same year at different places were decidedly helpful. At first the New England Water Works Association was the only strong advocate of uniform accounting, but other organizations followed its lead, the Bureau of the Census helped the work along, public-service commissions co-operated, and now waterworks accounting is beginning to look businesslike in most cities of 30,000 population and over. Below that limit the opportunities for improvement are still very great.

Recently the same thing has taken place in the sewerage field. The cost of sewers and sewage-treatment works has become a heavy burden on every community, and the importance of economy without any sacrifice of efficiency has been brought home to municipal officials and their engineering advisers. Unfortunately, the experience of one city is rarely available for much use in another city, financially speaking, because of a lack of any uniform system of accounting. Eight years ago the sanitary section of the Boston Society of Civil Engineers recognized the importance of the subject and adopted certain forms for reporting statistics, and two years later it published statistics from 24 cities arranged according to this system. It was often commended by the Census Bureau and eventually adopted as the basis for the collection of the bureau's information. Last year the bureau published its first volume of sewerage statistics, and after examining this critically a committee of the society has reported that the work was so well done that the society should abandon its endeavors along the same line. This is a high tribute to the work of the bureau and to the confidence felt in its willingness and ability to enlarge and strengthen the scope of its sewerage studies.

But it is to be hoped that the engineers in charge of sewerage systems will not be content to let their accounting methods become lax when they relate to items not called for in the bureau's schedule. If the sewage is screened, the volume or weight of the screenings and the size of the openings in the screens should be recorded, and a statement made of the method of disposal. A record should be kept of the volume of grit removed annually from the grit chambers and of the method of disposing of it. Such quantitative details of every step in the treatment of sewage, even if kept only approximately, become important when improvements are necessary, for they furnish the definite information about what has been done, which is essential in studying a project. The usual information which is recorded is of insignificant value and is occasionally so general as to be actually misleading.

Subway Car Design

WHEN the original New York subway was opened a group of engineers was extolling the achievements of the chief engineer and his assistants, whose ability and energy had made the marvel possible. Finally an electrical engineer in the group broke in and asked whether the mechanical and electrical engineers did not deserve some credit for the development of devices which allowed eight-car trains to be started and stopped in an incredibly short length and to be propelled with safety through the tube at speeds of 35 and 40 miles an hour.

The answer was obvious, but in discussing the new subways at New York, while much attention has been given to the civil engineering features, the electrical and mechanical equipment has been less in the public prints and in the public eye. But the mechanical and electrical engineers have not been idle. One of the chief evidences is the remarkable design developed by the New York Municipal Railway Corporation for its cars. Such has been the progress since the first subway was built that the new cars will allow an increase in passenger capacity per track of 25 per cent as compared with the original line. This improvement has not been made in one leap from the original subway car, since the equipment for the Cambridge subway marks an important intermediate step. The present subway cars are 52 ft. long and carry in the rush hours 169 passengers each. The new car is 67 ft. long and has a maximum carrying capacity of 270. The additional carrying capacity has been secured not only by increasing the length, but by increasing the width from 8 ft. 6¾ in. to 10 ft. and by arranging the seats in four groups divided by three pairs of doors in each side. One pair of doors is at the center of the car and the others at the quarter points. The wastefulness of the end vestibules has therefore been eliminated. At the same time the design has been so worked out that the cost per car is little more than that of its predecessor, while the weight per passenger has been decreased by an amount varying from 83 to 145 lb., depending upon the combinations of motor and trailer cars used. An interesting arrangement of both transverse and longitudinal seats has been worked out after most thorough study, taking into consideration the proper proportion between standing room and seats and the influence on time of stopping of the distance the passengers are from the doors.

Summarizing the advantages of the new car the "Electric Railway Journal" in a recent description said that the new design as compared with the present cars "will increase the ultimate carrying capacity of the tracks by 20 to 25 per cent, save about $200,000 a year in platform costs (wages of trainmen), save about $2,000,000 on an order for 600 cars and from $1,000,000 to $2,000,000 in power-system capacity because of the decreased energy consumption per passenger carried." With these figures in mind the part that engineers other than civil play in the development of rapid-transit facilities is not apt to be lightly regarded.

Developments in Pier Construction

THE great and increasing dimensions of transatlantic passenger steamers have far outgrown the existing facilities and reasonable potentiality of the piers in the lower North River, New York, where they have heretofore been accommodated. The Department of Docks has, therefore, established a new location for them in the vicinity of Forty-second Street. Owing to the outshore limit for all construction presented by the pierhead line established by the War Department, the very long piers and their adjacent wide and deep slips must extend within the original shore line, where heavy rock excavation is necessary, while at the outer end of the pier the river bottom, of soft mud, 150 ft. deep, affords very poor bearing for foundations on which heavy loads are imposed.

The maximum draft of 38 ft. provided for requires, with due consideration of extreme tide variation and the character of the bottom, a depth of 44 ft. below mean low water in the rock-bottom slips 360 ft. wide. Conditions of diametrically opposite kinds of severity, not previously encountered together for such structures, were thus unavoidable for the very large and costly Forty-sixth Street pier, described on page 120. They have been met by a combination of equally different types of construction in one pier, which has a substructure partly of rockfill and concrete walls on rock bottom and partly of a timber platform on timber piles in deep water and soft mud. Both are designed to maintain their stability and alignment and support a two-story steel shed with the necessary loads.

The solid-fill portion of the pier, although not a new type, is a rare one, and has not been previously constructed in New York. The difficulties of building it and the adjacent slips in deep water were met by the construction of an enormous cofferdam, described in this journal July 18, enabling this portion of the site to be unwatered and securing greater certainty, rapidity and economy in the work. This enables the pier foundations to be carefully built, assures the absence of any dangerous rock projections in the bottom of the slip, and enables the walls of the latter to be advantageously finished with a smooth vertical surface, very satisfactory for wharfage.

For the outer end of the pier, in mud so soft as to have no direct bearing value, a capacity of 15 tons per pile has been secured by using piles of the greatest length obtainable and providing them with lagging to develop increased skin friction. Although the lagging method was so successful in the $25,000,000 Chelsea piers that the unavoidable settlement was reduced to 3 in. in five years, and so regular as to cause no distortion or apparent change, these results were not accepted alone but were supplemented by long-time full dead and live load tests of actual foundation pile clusters. These have verified and established pile capacities for the particular conditions and have determined important features in the treatment of the superstructure, justifying, among other things, an elimination of heavy concrete floors and thereby effecting a very considerable saving of cost and weight by the adoption of a thin reinforced-concrete covering for a plank floor. Fireproofness is complete in the rock-filled substructure and is approximated in the pile substructure by the concrete floor surface and the reinforced-concrete fire walls below the floor. The design has been developed to promote the utmost durability and permanence, and special attention is given to the foundations for the steel superstructure, which are carried down to pile tops at low-water level and may have special provisions to meet the possible deterioration of concrete cast in salt water.

Both in the design and in the plans for its execution a degree of thoroughness has been shown which will make the structure a subject for fruitful study by those having difficult pier work before them.

Utilization of Solar Energy

FOR more than a quarter century the direct utilization of the sun's rays for power production has interested the physicist. Experimental progress along this line has made rapid advance during the past four years, and while the commercial application of solar energy is still in embryo, some interesting apparatus has been developed by engineering investigators. One of the most comprehensive papers ever prepared on this subject was presented at the April, 1914, meeting of the Society of Engineers (London) by A. S. E. Ackermann, whose pioneer work in this field in Egypt, in association with Prof. C. V. Boys, F.R.S., in 1913, represents perhaps the most extensive development thus far attained in this suggestive branch of research.

The problem of solar heat utilization is two-fold. The production of an absorber of efficient and reliable design is the first essential, and the utilization of the steam or other vaporous energy collected is the second. A fair measure of success was attained with the apparatus built on the Shuman design at Tacony, Pa., in 1911, where 0.09 lb. of steam was produced per square foot of solar radiation per hour, the plant occupying 0.41 acre, with a total boiler heating surface of 5005 sq. ft., 10,296 sq. ft. of solar radiation being collected. The absorber was of the cellular type, with concentration by glass mirrors upon flat boilers mounted behind sheets of window glass. In this apparatus the heating effect was largely due to the properties of glass, which permit the sun's radiation to pass through to the boiler without conducting away the sensible heat from the glass on the side next the boiler. The thermal efficiency of the Tacony absorber was 29.5 per cent for the best run of one hour at atmospheric pressure. The Shuman-Boys absorber used in Egypt in 1913, however, gave 0.12 lb. of steam per hour per square foot of sunshine collected, evaporation being from and at 212 deg. Fahr. This apparatus had a total area of solar radiation collected of 13,752 sq. ft., a total boiler heating surface of 2819 sq. ft. and occupied 0.88 acre. The thermal efficiency was 40.7 per cent, due in a large degree to the improvement in design. In the Egyptian installation the boiler units were U-shaped and situated in the focus of parabolic silvered-glass mirrors, which were automatically turned toward the sun during the runs. The quick steaming resulting closely parallels modern fire-engine practice.

The steam generated by solar energy in plants designed for direct utilization has generally been of low pressure, although several American investigators have successfully produced high-pressure steam on a moderate scale from solar apparatus. In the work of Mr. Ackermann and his associates specially designed low-pressure engines were developed and pumping loads were handled. On account of the poor condition of the engine end of the plant, the combined absorber and engine tests in Egypt do not fairly indicate the painstaking development which has been accomplished. For the details of these interesting engines the reader must consult the paper, which discusses the problem of design in thoroughgoing fashion and also points out many salient conclusions with respect to the effect of wind, humidity, clouds, condensation requirements, proportions of absorber boilers, steam pressures and other conditions upon performance. In most of the small-engine tests the efficiencies obtained were excellent in comparison with results obtained in testing simple and compound condensing engines of larger sizes, and if the present status of prime movers suited to sun-power plant service is still far from standardized, there appears to be no insuperable obstacle to the commercial utilization of steam generated in absorbers of the types above mentioned.

It is no doubt too early to weigh the merits of such plants in the balances of industry, but their low cost of operation in countries characterized by long continued sunshine, high fuel prices and vital need of irrigation suggests the desirability of their development under similar conditions elsewhere. The Egyptian experiments constitute an excellent starting point for further research.

Largest Steamship Pier on Manhattan Island

Rock Foundations and Long Mud-Bearing Piles Combined in West Forty-Sixth Street Structure, Adjacent 1000 x 360-Foot Slips 44 Feet Deep

GREATER dimensions and a greater depth of water in the adjacent slips than for any other pier along Manhattan Island characterize the new municipal pier in the North River at the foot of West Forty-sixth Street, New York City, now under construction by the Department of Docks and Ferries, for the accommodation of the largest steamers yet built or proposed. The pier will be otherwise unusual in that the outer end will have a substructure of the standard wooden-pile type developed and improved by this department, while the inner end will have a substructure of solid fill between concrete retaining walls carried to rock. Preliminary contracts amounting to about $627,000 have been awarded and the contracts for the pile substructure and the two-story steel superstructure will, it is estimated, amount to from $250,000 to $300,000 and from $600,000 to $700,000 respectively, bringing the total probable cost of the pier up to $1,500,000 or more.

Other features of special interest are the vertical walls at the inner ends of the slips, the 40-ft. spans between superstructure columns at the shore end of the pier, the great length of piles at the river end—which do not reach hard bottom and are supported chiefly by skin friction—the concrete column pedestals with pile foundations, the permanent and temporary main floors, the method of constructing the rock foundation and excavating rock from the slips in the dry and the cofferdam of unprecedented dimensions and special construction. The cofferdam was described in the Engineering Record of July 18, page 58.

Preliminary Work

An area of about 300 x 930 ft., reaching across the shore end of the pier and slips, will be inclosed by a cofferdam now approaching completion, and will allow all of the work except the earth dredging and the pile driving to be done in the dry. This cofferdam, shown in the general plan, consists mainly of a series of pockets generally 24 ft. long and 16 ft. in minimum width, with their inner and outer faces curved to a radius of 24 ft. They are made of interlocking steel sheet piles driven to rock. The pockets are filled with earth, chiefly dredged inshore of the cofferdam, which, in combination with the heavy joint tension produced by the internal pressure, is expected to insure sufficient watertightness, while stability is provided by the resistance offered by about 99,000 cu. yd. of riprap banked around the inside of the cofferdam.

At the southwest corner of the cofferdam, where an existing crib pier diminished the requisite clearance for the slip excavation so much that stability could not be provided by the wide interior riprap bank, a special corner construction was adopted and stability assured by changing the shape of the pockets, increasing their diameter to 55 ft., and filling them with riprap instead of earth. In other words, the riprap was transferred from the inner face of the cofferdam to the interiors of these pockets.

At the north and south ends of the cofferdam the pocket construction is supplemented by an extension, inshore, of single lines of steel sheet piles; some of these are anchored to adjacent cribbing and the space between them and the cribbing is filled with puddled material. It is estimated that the pockets will require 23,000 yd. of earth filling and that after the completion of the pier the demolition of the cofferdam will involve the removal of about 123,000 yd. of riprap and earth. An embankment of miscellaneous fill is built against the outer side of the cofferdam to offset any tendency of the latter to lean outward.

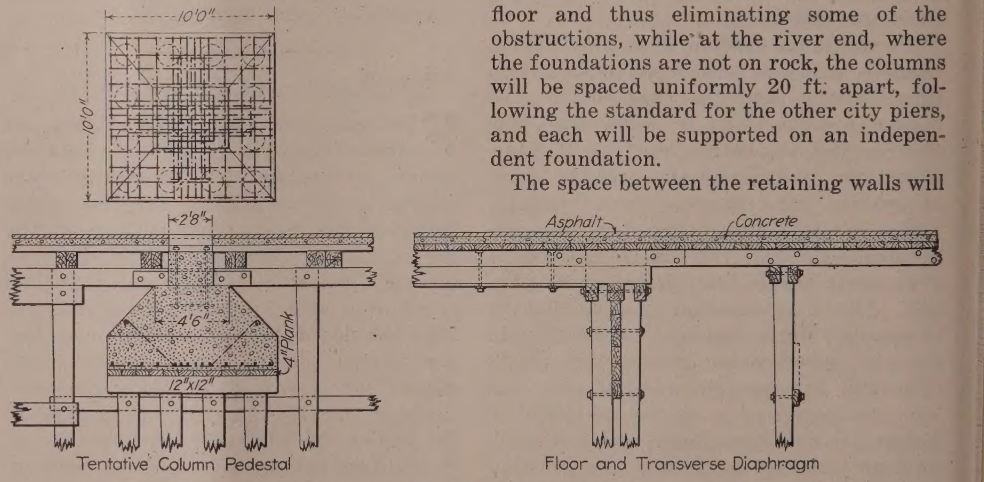
Provision is made for filling the cofferdam if necessary by means of two wooden sluiceways through the steel pockets and riprap on the river side of the cofferdam. These sluiceways are 12 ft. wide, 7 ft. deep and 25 ft. long inside the steel piles. The outer end of each is closed by a full-width vertical sliding gate 4 ft. high and 6 in. thick, closing on the lower edge against a rubber gasket and a concrete sill. The gate is operated by chains at each end wound on a 3-in. transverse shaft driven by geared cranks. When in the closed position it is wedged in place.

Rock Foundation Pier

At the shore end 219 ft. of the pier are built on the natural sloping surface of the rock, which is from 15 to 45 ft. below datum—mean low-water level. As the wide slips on both sides of the pier at this point are excavated to a depth of 44 ft. below datum, the rock excavation on the shore ends and sides of each slip will be carried down with vertical faces made by channeling machines or by drilling and broaching. On the pier and bulkhead lines walls of massive concrete alongside this excavation will be carried up from the surface of the rock as indicated in the accompanying typical cross-sections. The front faces of these walls will be vertical and their rear faces will be battered by rectangular offsets 1½ ft. wide and 4 ft. high. The walls will have a uniform width of 4 ft. on top and the maximum height will be about 58 ft., with a corresponding base width of 23 ft. The upper part of the wall and the surfaces will be of plain concrete, but the interior and lower portion will be of cyclopean concrete. The outer faces of the north and south walls will be protected by wooden fenders.

Between the walls there will be two longitudinal rows of concrete column piers, 40 ft. apart longitudinally, to support the columns of the two-story steel pier sheds. At the shore end these columns will be connected by longitudinal girders carrying the intermediate transverse beams of the second floor and thus eliminating some of the obstructions, while at the river end, where the foundations are not on rock, the columns will be spaced uniformly 20 ft. apart, following the standard for the other city piers, and each will be supported on an independent foundation.

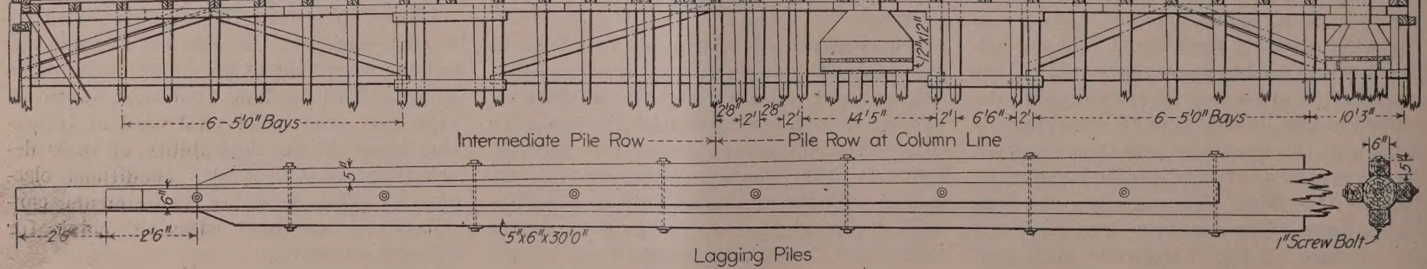
The space between the retaining walls will



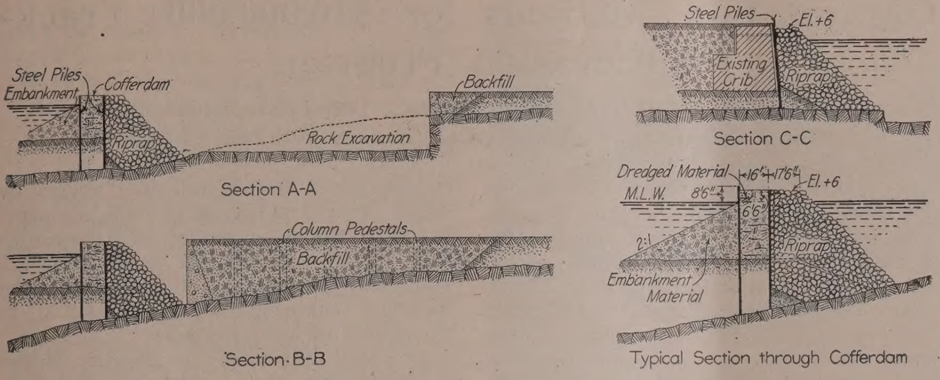
Tentative Column Pedestal

Floor and Transverse Diaphragm

WOODEN-CONCRETE DECK AND SUGGESTED PEDESTAL CONSTRUCTION



HALF-TRANSVERSE SECTIONS OF PILE SUBSTRUCTURE AND DETAIL OF 89-FOOT LAGGED PILE



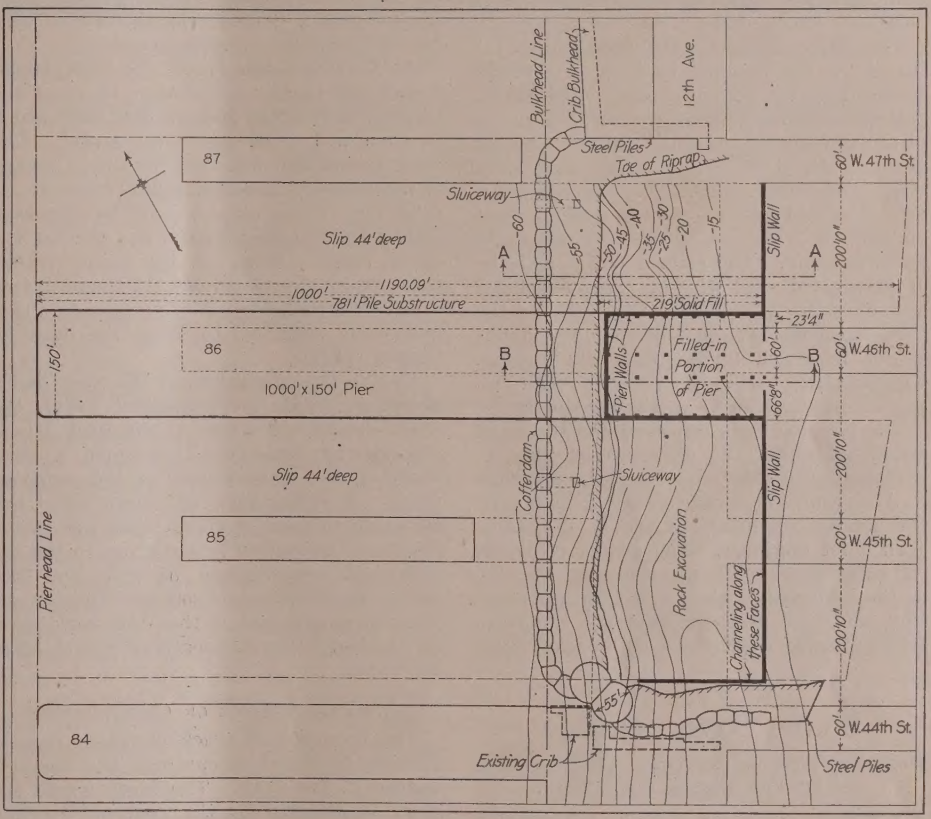
TYPICAL SECTIONS OF PIER AND SLIP AND COFFERDAM DETAILS

be filled to within about 10 ft. of main floor level with rock excavated from the adjacent slip and covered with 5 ft. of riprap supporting 5 ft. of earth or cinder filling. This filling will be accurately crowned and on it will be laid 6 x 10-in. yellow-pine sills about 2 ft. apart, to which will be spiked the temporary floor of 3 x 10-in. yellow-pine planks. After the fill has thoroughly settled the temporary wooden floor will be replaced by the permanent concrete-and-asphalt floor. Two 4-ft. sewers will be carried longitudinally through the pier under the floor and two standard-gage tracks will be laid on the floor in the center of the pier.

PILE SUBSTRUCTURE

The outer end of the pier, for a length of 781 ft., will be built on a substructure consisting of about 7000 pine or cypress piles 16 in. in diameter at the butt in the outer end of the pier and 14 in. in diameter elsewhere, and with tips not less than 5 in. in diameter. The piling plan is divided into zones according to the required length of the piles, which, at the outer end of the pier, is the maximum procurable, varying from 85 to 89 ft.

In about three-fourths of this portion of the pier the piles do not reach hard bottom, and in order to increase their bearing capacity due to frictional resistance, which in many cases is their total value, about half of them are lagged on four sides for 30 ft. at the lower end, as indicated in the details. All vertical piles are to be driven with a 3000-lb. hammer having a concave face and falling 10 ft. for lagged piles and 12 ft. for plain piles. Plain piles are to be driven to hard bottom or to a penetration of not more than 1 ft. in the last ten blows. Piles in the lagged zone are to be driven to a penetration of not more than 4 ft. with the last ten blows or as directed by the engineer.



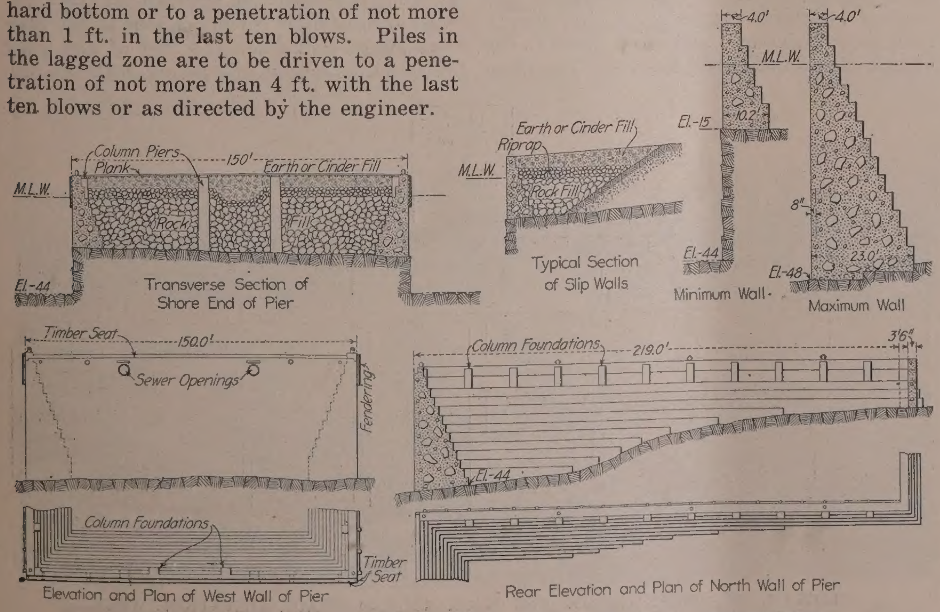
CONTOUR MAP OF PIER, SHOWING COFFERDAM AND ROCK EXCAVATION

The piles are driven in seventy-five transverse rows. The outer end of the pier is strengthened by making three of these rows double and bolting 6-in. plank diaphragms 7 ft. deep between the piles at their upper ends as shown in the details. Additional piles are driven in the outer corners, which are rounded and braced in the usual manner.

The regular pile rows are spaced 10 ft. apart and the piles in them are arranged with clearance for two lines of box sewers. Alternate rows are provided with diagonal bracing above water level and one spur pile at each end. In the intermediate rows these are omitted and a special arrangement is introduced to provide for four column pedestal foundations having twelve piles for each end pedestal and sixteen for each intermediate pedestal. In general the piles in the pedestal clusters are alternately plain and lagged, and where two piles are close together in the transverse rows one of them is plain and the other is lagged. The remainder of the single piles in the lagged zone are nearly all lagged. The piles in the pedestal clusters are proportioned for a maximum load of 15 tons each on lagged piles.

The concrete column pedestals are seated on grillages on top of the foundation piles, and a tentative design for them, here shown, calls for reinforcement with 1-in. longitudinal and transverse bottom rods with 1-in. anchor rods on each side of the beveled face of the pier, connected to them by ½-in. inclined ties. These details are still subject to change, however, and an I-beam grillage has been under consideration. All joints in the framing, all places where timbers are in contact at intersections, and all side caps, rangers, chocks and timbers on the upper surface of the wooden floor, except where the latter is asphalted, are freely treated with carbolineum.

The wooden substructure is divided into



CONCRETE WALLS AND CHANNELED FACES OF ROCK EXCAVATION three parts by two fire-resisting transverse walls 8 ft. high to the bottom of the floor. The lower half of the wall, down to low-water level, consists of a diaphragm of 4-in. horizontal planks capped by a 6 x 6-in. timber supporting a vertical concrete slab 6 in. thick, reinforced with expanded metal.

Longitudinal stringers are laid on the pile caps and covered with a closed deck of 4 x 10-in. yellow-pine transverse planks. An upper course of 7 x 12-ft. timbers laid flat on the 4-in. deck is built around the outer edges of the pier and has a width of 6 ft. The vertical inner edge of this course is faced with 7-in. channels, forming a curb provided with horizontal 4 x ¼-in. carriage bolts through the webs, 2 ft. apart on centers. Steel curbs of the same height are also placed around openings left through the 4-in. deck for elevator pits and fire manholes.

The entire surface of the wooden deck inclosed by the channel curbing, except as occupied by the column seats, elevator pits and fire manholes, is covered by 4¾ in. of reinforced concrete laid in transverse strips about 5 ft. 8 in. wide, alternating with strips about 6 ft. wide. The narrow strips are to be laid first and are to become hard enough to walk on before the alternate strips are laid. The reinforcement must be of galvanized steel wire or expanded metal and must be attached to the bolts in the curb channels. It must be in pieces about 4½ ft. wide and not less than 100 ft. long. The concrete will be covered with an asphalt-wearing surface 2¼ in. thick.

The pier has been designed and the work conducted under the direction of the Department of Docks and Ferries, of which R. A. C. Smith is commissioner, R. C. Harrison deputy commissioner and C. W. Staniford chief engineer. The contract for the cofferdam and rock excavation was awarded to the Holbrook, Cabot & Rollins Corporation, and it is expected that the contracts for the timber work will soon be advertised.

Snake River Bridge

Single-Track Viaduct Structure 3920 Feet Long and 280 Feet High at Perry, Wash.

ON the front cover is shown an erection view of the bridge being built by the Oregon-Washington Railroad & Navigation Company across the Snake River at Perry, Wash. The bridge is the connecting link in the Spokane-Ayer line and will shorten the present route 50 mi. It is a single-track structure 3920 ft. long and 280 ft. above the river bed. The south approach is on a 6-deg. curve with a 240-ft. spiral.

The river piers were built with pneumatic caissons, the original plan of using open cofferdams having been abandoned when bedrock was found to be overlaid with from 20 to 40 ft. of gravel and boulders. Concrete for the south approach was placed from an aërial cableway 800 ft. long, approximately tangent to the curve midway between the river and the end abutment. The truss spans were erected as cantilevers from the north approach by a jib traveler which rested on the top chord of each truss.

The substructural work was done by the Caughren-Boynton Company and the Missouri Valley Bridge & Iron Company. The superstructure is being erected by the Kelly-Atkinson Construction Company, of Chicago, of which J. E. Baggs is superintendent.

Organization Methods for $10,000,000 Track-Elevation Program

After Careful Preliminary Study Rock Island Railroad Adopts Unusual Schemes on Its Heavy Construction Work near Chicago

A CO-OPERATIVE spirit fostered by careful study and recognition of merit has built up for the Chicago, Rock Island & Pacific Railway a track-elevation organization that was able to report up to June 1, 1914, a remarkable gain over the lowest contract figure on its grade-separation work being done with company forces near the southwest limits of Chicago. The work for 1914 lies between Seventy-second and Eighty-fifth Streets and is part of a $10,000,000 track-elevation program which the Rock Island is expected by the city of Chicago to complete in seven years.

At Seventy-ninth Street the Rock Island tracks are being carried over those of the Chicago & Western Indiana Railroad, which in turn will cross above the street. The Rock Island now has three and the Chicago & Western Indiana four main tracks at the crossing. These numbers will be increased to five for the Rock Island and ten for the other road. Owing to the heavy traffic, there being more than 500 train movements a day, a complicated construction program has been required to keep the lines in proper service.

Due to the magnitude of the work and a desire to make every moment count, the track-elevation division of the Rock Island engineering department occupied a year studying the many phases of the proposed work. A comprehensive study of track elevation in various large cities was made, particular attention being given to the design and construction of walls and subways, the handling of concrete and the organization problem so that the work could be handled without interference with train operation.

GENERAL SCHEME OF ORGANIZATION

The engineer of track elevation reports directly to the chief engineer and has sole charge of the work. The execution of the work is directed by the general foreman, who reports directly to the engineer of track elevation, as does the chief clerk, cost engineer, assistant engineer and assistant trainmaster. Each of these officers has various assistants reporting to him, who in turn have assistants, and so on down to the gang and the individual laborer. The illustration shows the organization scheme in detail. It will be noticed that the engineering party has no connection with the execution of the work, except to see that the plans and specifications are followed. The assistant engineer has duties similar to those of a resident engineer.

In order to reward the efficient laborer a graded wage scale, something unusual in railroad work, has been adopted. By means of this scale the gang foreman has certain pay limits assigned to him. He can then pay his laborers according to their various abilities. The scheme has to date acted as a great stimulus to the men to prove their worth. Each gang has an assistant called a pusher who watches the men. The foreman not only watches the men but also any machine operations under his care. The work itself is divided into fourteen sections.

ACCOUNTING SYSTEM AND UNIT COSTS

A cost engineer, who is rated an expert engineer, accountant and analyst, has charge of the working accounts and through him unit costs are obtained. The work is divided into sixteen accounts, as follows: (1) Concrete, (2) excavation, (3) fill, (4) track, (5) bridges, (6) train service, (7) supervision, (8) engineering, (9) concrete piles, (10) wood and steel movable forms, (11) reinforcing, (12) sheeting, (13) waterproofing, (14) miscellaneous, (15) machinery and equipment and (16) steelwork.

Each gang foreman hands in a report of his specific task every day. This report indicates the location, occupation, rates and hours of each laborer and the work done. The cost engineer then takes these reports and figures the labor, additional, total and unit costs. These unit costs are then posted under the proper accounts on monthly summary sheets, which give the total quantity in any section. Then a summary is kept of all the sections for each day. All accounts are closed each day and the unit costs obtained for any feature of the work in any section.

By this system the engineer of track elevation can follow the operations of the various gangs and when the cost of any gang is found to be out of keeping with the pre-

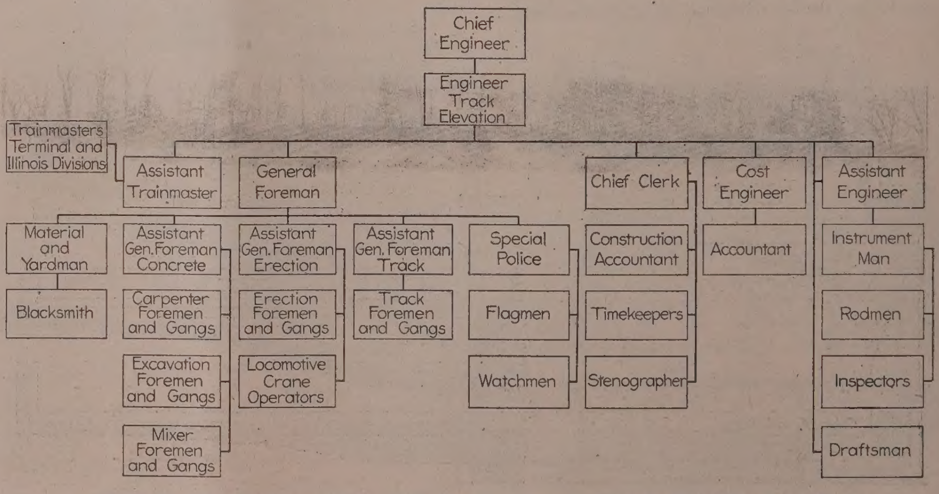
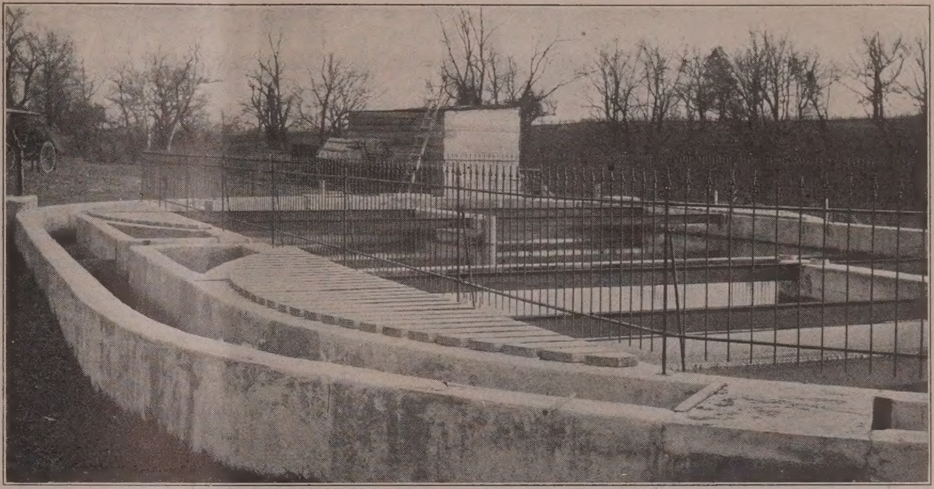


DIAGRAM OF ORGANIZATION FOR ROCK ISLAND TRACK-ELEVATION WORK vailing unit cost for its particular kind of work he receives a warning from the foreman's report sheet. As an example, at the end of each day the engineer of track elevation can tell exactly what the concrete mixers have accomplished and their cost of operation. Whenever he finds costs high he sends the cost engineer out to the part of the work in question to study the operations. When the latter discovers what he considers is wrong, he calls the foreman of that particular feature of the work and points out the situation. The foreman then, if convinced, gives the order that will provide the results needed. Like the other engineers, however, the cost engineer may not give orders to those who execute the work. In other words, he co-operates without any interference or use of authority. In case the cost engineer cannot convince the foreman or his superiors, and he still has faith in his idea, the matter is taken to the highest court, that of the engineer of track elevation. A progress report is made each week which shows the percentage gain in each section on whatever accounts are kept in that section.

FIELD OFFICE

The execution of the work is handled in the field from a 40-ft. office car which houses the entire accounting, clerical, engineering and executive departments. Representatives of the regular accounting and purchasing branches have their headquarters in the car. By this plan accurate and thorough reports are made without the usual duplication. Stock material can be secured by this field office within a day's notice by special telephone to the system storehouse at Silvis, Ill. In this manner overstocking is avoided and second-hand material is used when circumstances will permit. All data, whether correspondence, plans or estimates concerning the work, are kept in the office car, and as a result the usual delay in routine matters and the unnecessary correspondence with a remote office are avoided.

The effectiveness of the organization is brought out by the fact that an extremely low rate for steelwork was obtained as a result of the rapid and machine-like way the work is handled. The man who took the steel-erection contract figured that he would make money if he could erect 100 tons a day. He is erecting about 300 tons a day. Not an accident or delay to a single train has occurred to date.



TWIN IMHOFF SETTLING TANKS AT CHAMBERSBURG SEWAGE-TREATMENT PLANT

Imhoff-Tank Operation at Chambersburg

Observations on Sewage-Treatment Plant Which Has Been in Service More Than Two Years in This Pennsylvania Town

By C. F. and P. E. MEBUS
Of Albright & Mebus, Consulting Engineers, Philadelphia

THE Borough of Chambersburg, Pa., has in operation a sewage-treatment plant comprising two concrete Imhoff settling tanks, dosing chamber, percolating filter, three secondary settling tanks and two sand filters for drying the sludge. It serves a population of 6000.

The sewer system of Chambersburg is of the separate type. The sanitary sewers all discharge into a main reinforced-concrete intercepting sewer laid along the Conococheague Creek, which flows through the borough. Sewage leaves the intercepting sewer and flows through an inverted siphon to the collecting well of the pumping station, from which it is automatically pumped to the Imhoff tanks. The designed capacity of the plant is about 900,000 gal. per 24 hr., although at present the flow never exceeds 500,000 gal. per day.

The pumping station contains two electric 6-in. centrifugal pumps directly connected to Westinghouse two-phase, alternating-current motors, which are started and stopped automatically by float switches. An auxiliary unit has also been installed consisting of a 6-in. centrifugal pump connected to a gas engine.

There are two open Imhoff tanks, each 36 ft. in diameter at the sedimentation compartment and 26 ft. in diameter at the digestion compartment. The apex of the inverted cone in the bottom of the tank is about 26 ft. below the water level. The sewage enters and leaves the tanks over four baffled weirs at each end of the tank, all of which are set at the same elevation. In order to distribute the sludge more evenly in the tanks, the direction of the flow may be reversed by pumping the crude sewage to either end of the tanks. A concrete channel is built alongside the tanks to carry the effluent to the dosing chamber. The sludge is removed by an 8-in. cast iron pipe extending nearly to the apex of the inverted cone in the bottom and operating under a 9-ft. head. The flow from the pipes is controlled by gate valves located in manholes adjacent to the tanks. A 10-in. sewer conveys the sludge to the drying beds, so that no manual labor is necessary to remove the sludge, except for opening the valves.

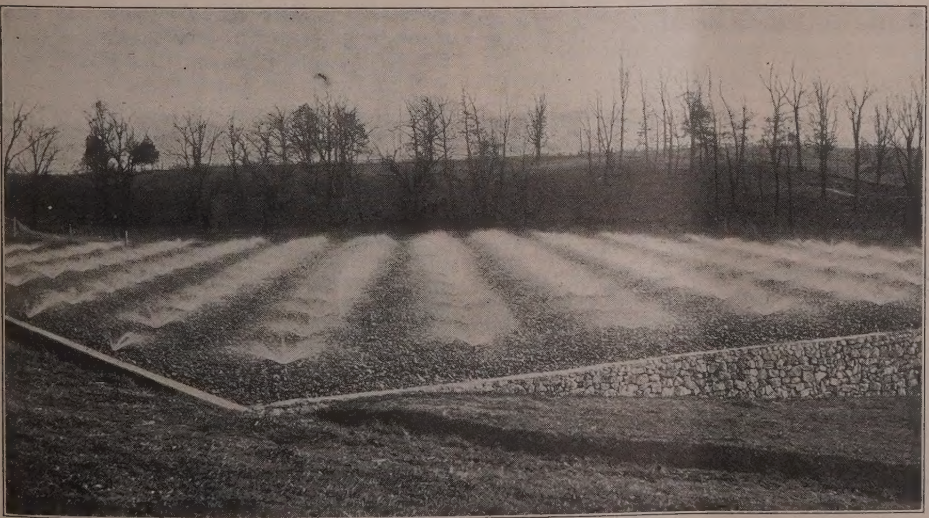
PERCOLATING FILTER

The percolating filter, which is 125 x 160 ft. in plan, contains 140 fixed nozzles of the Taylor hexagonal type. The filtering media are 1½ to 3½ in. in size, limestone about 7 ft. deep and underdrained by 5-in. cement underdrains laid on a concrete floor. The effluent is collected in the main conduit and carried to the secondary tanks, of which there are three, and which together provide a sedimentation period of about 1 hr. The effluent of these tanks is conducted to the deeper water of the creek without further treatment.

There are two sand beds for drying the sludge, each 45 x 60 ft. They are divided by a concrete wall so that one bed may be used independently of the other. The sand, which is 2 ft. deep, is underdrained by a system of vitrified pipes which carry the drain water to the creek.

OPERATING RESULTS

The Imhoff tanks at Chambersburg have now been in operation long enough to give a fair idea of what may be expected of



PERCOLATING FILTER AT CHAMBERSBURG, WITH THE 140 NOZZLES OPERATING them. The sewage reaches the plant in a fresh state and all efforts are made to keep it so and to pass it through the plant as rapidly as practicable. There are no large vent chambers in the centers of the Imhoff tanks; the side vents, however, are quite large. The vents in the centers of the tanks consist of eight 6-in. pipes, extending from the decomposing chamber to about 15 in. above the surface of the sewage. Little gas is given off and only a small amount of scum is formed in these pipes. The side vents, however, are decidedly active and gas is given off in large quantities. This gas burns with a pale-blue flame characteristic of methane, and the percentage thereof in the entire volume of gas is undoubtedly quite large. It is entirely odorless and therefore inoffensive, due to the fact that its main constituent is the odorless methane. H_2S odors were not noticeable at the Imhoff tanks.

Scum

Scum due to improper operation has reached a thickness of about 12 in. in the vents of one tank and about 9 in. in the other. This scum has been entirely removed several times, placed on the drying beds and spread on land after drying. This method although removing all of the scum does not prevent its formation, which is undoubtedly the logical way to deal with this problem. Daily efforts made to force this scum to sink are futile, due to its large gas content. There is now under construction a pressure water system by which it is proposed to break up the scum while forming, and thereby prevent the larger accumulations in these vent chambers. The scum in both vents was decidedly black in color, fibrous, porous, and its odor was that characteristic of damp earth. At the time of a recent inspection by the writers the surface of the sewage in the sedimentation compartment was practically free of scum and other floating matter, except some soap and grease balls which had formed at the weir baffles. Such floating matter is removed once weekly.

The sludge digestion compartments have a combined capacity for storing the sludge accumulation of 12,000 persons for a period of about four months. Sludge has been withdrawn a number of times during the past two years. It is always a semi-thick fluid, containing much gas and flows readily through the pipe sewer to the drying beds. When removed it is black in color, although upon drying it is found to have a deep brown color. Its odor, that characteristic of damp soil, is so slight that in order to perceive it at all one must get very close to it. It is a homogeneous mass and its original constituent parts are no longer discernible. It is evident that such sludge cannot cause any nuisance when spread out to dry on the sand beds. Samples of the sludge were analyzed by the testing laboratories of the city of Philadelphia, with results as shown in Table 1. This table shows in a parallel column the results of sludge analyses as made by the Emscher Federation.

The writers have conducted a number of tests for settleable solids with the conical settling glasses, as used in Germany, the results of which are given in Table 2. Again, as a matter of comparison, there are shown the results of tests of settleable solids at plants of the Emscher Federation. The samples were allowed to stand about 1½ hr. for settling, this being necessary due to operation of the pumps. The samples of the effluent have been collected 1½ hr. later than those of the influent.

The sludge up to this time has been given to the farmers living near the sewage plant, who have spread it on their land. They seem well pleased with it, and expressed the opinion that it benefited their crops. They also expressed a willingness to dispose of the sludge produced at the plant in the future.

TABLE 1—ANALYSIS OF SLUDGE SAMPLES

Location of plant	Dry residue, percentages Mineral	Volatile	Nitrogen	Fats
Rechlinghausen	58.51	41.49	1.65	5.28
Essen N. W.	46.65	53.35	1.17	3.39
Bochum	64.43	35.57	1.24	6.91
Chambersburg	58.38	41.62	....	3.22

Methylene blue samples are also collected and incubated. An improvement in the effluent of the Imhoff tanks over the influent is always shown by the longer incubation period required by the former. The effluent of the percolating filter and also the final effluent have always retained blue color at the end of 5 days, which meets the State's requirements.

Conclusion

The Imhoff tanks as operated at Chambersburg have, in the writers' opinion, borne out the claims of their inventor. The sewage is kept fresh and free from the "blown up" sludge, the operation is practically odorless and the sludge produced is odorless, dries quickly and its ultimate disposal is easily and satisfactorily accomplished. It is the opinion of the writers that the operation of the percolating filter is also more satisfactory, due to the comparatively lesser amount of solid matters in the tank effluent, which tends to avoid undue clogging of both nozzles and the filter media. Imhoff tanks which are correctly and carefully designed and intelligently operated will undoubtedly give operating results similar to those obtained by the Emscher Federation, which has so completely revolutionized the art of sewage treatment.

TABLE 2—TESTS FOR SETTLEABLE SOLIDS

Location of plant	Settleable solids (average figures). C.C. per liter Influent	Effluent	Per cent. Reduction	Detention Periods
Rechlinghausen	7.17	0.5	93	Appx. 60 min.
Bochum	4.0	0.2	95	Appx. 65 min.
Chambersburg	4.9	0.2	96	Appx. 90 min.

The sewer system and treatment plant at Chambersburg, Pa., were designed by Charles F. Mebus, of Albright & Mebus, consulting engineers, of Philadelphia, Pa. The work was done under the immediate supervision of R. M. Huber, formerly borough engineer. J. Hayes Mowery, superintendent of the electric-light department, had complete supervision of the treatment plant, and J. Speer is chief operator.

CORPORATION COCK LEAKS in Milwaukee water mains have been reduced by a change in the style. In place of a straight tail piece, one with a 45-deg. curve is used. By bending the lead pipe into an inverted gooseneck the lead pipe rests on solid material and the weight from above cannot rupture the pipe or cock. Many services had to be dug up in the past on account of leaks, caused by the pressure splitting the lead pipe or breaking either the thread in the top inserted in the main or the shoulder on the tail piece.

Preventing Village Fires

Building Ordinance for Small Towns Recommended by the National Board of Underwriters

THE National Board of Fire Underwriters, of New York City, and the National Fire Protection Association, of Boston, have, through a committee on construction of buildings, published an ordinance commended to small towns and villages, providing for fire limits and the construction and equipment of buildings. It is an abbreviation of modern requirements representing best practice in building construction and has been compiled with special reference to sections of small towns. It is designed to arrest present hazardous practices and to serve as a reasonable regulation of ordinary building construction in communities where congestion of values is not abnormal. Numerous building experts have advised in its preparation and much effort has been devoted to make it brief in text and broad in scope.

Among the important features of the ordinance are sections providing for the construction of incombustible walls, cornices and roofs. Within the stated fire lines the limit of height is four stories, or 65 ft., except for fireproof construction, which may be ten stories, or 125 ft. The limit of floor area between fire walls of non-fireproof buildings is 5000, 6000, or 7500 sq. ft., according as they front on one, two or three streets. The area is extended 50 per cent for fireproof buildings and 50 per cent for other buildings fully equipped with automatic sprinklers.

Shafts

In all buildings except private residences stair shafts and elevator shafts shall be separately inclosed by incombustible partitions; exits are carefully specified and general requirements are provided governing areaways, bay windows, chimneys, coal pockets, concrete construction, cornices, doors, electric installations, partitions, shutters, fire traps and walls, strength of materials, lath-and-plaster construction, parapets, skylights, stairs, terra-cotta construction and the general equipment and installations in building.

A horizontal exit as a means of rapid and safe egress through a fire wall is strongly recommended. The wall or partition should be built of fire-resisting material not less than 3 in. thick, be securely attached to the walls, floor and ceiling of the room which it subdivides and be provided with one or more self-closing fire doors. Each area must be sufficient to accommodate all employed upon the floor and must be provided with at least one independent exit to the street. A doorway in such a partition or fire wall may be considered the equivalent of three times the same width of stairway.

The use of a smokeproof tower or stairway is recommended as one of the best-known means of safe escape from a burning building. This also furnishes a protected position from which firemen can attack a fire on any floor. Such a tower is built entirely of incombustible materials and has no direct openings into the interior of the building it serves. It is reached by a fireproof open-air balcony or interior open-air vestibule, thus effectually excluding smoke and fire from the tower.

Correspondence should be addressed to Ira H. Woolson, consulting engineer, National Board of Fire Underwriters, 135 William Street, New York City.

Underpinning New Buildings on Swampy Soil

Reinforcing Wooden-Pile Foundations in Muck with Steel Piles Driven to Rock by Steam Hammer and Hydraulic Jacks

IRREGULAR settlements of the foundations of two large apartment houses in the Borough of The Bronx, New York City, have cracked the walls so badly and thrown them so much out of plumb that they were in danger of being totally wrecked before being occupied. The settlement has been arrested and it is expected that it will be entirely stopped and safeguarded and that the buildings will be raised, plumbed, and made stable by the completion of the work now in progress, which consists of shoring the walls on needlebeams and reinforcing the old pile foundations by new steel piles driven to rock and capped with steel beams and with continuous longitudinal reinforced-concrete girders bearing on both old and new piles.

The site at Ellsmere Place and Crotona Parkway is partly in a recent fill and partly on the dry bottom of an old pond, where the soil consists of 12 to 14 ft. of black muck and decayed vegetable matter, 4 to 6 ft. of silt and 2 ft. of small boulders overlying the rock surface, which slopes about 1:10. The ground-water level is very near the surface. The buildings are two five-story brick apartment houses, together occupying an area of about 100 x 200 ft. There are no columns, the floors bearing on the exterior and partition walls, which have foundations of rubble masonry laid in cement mortar with concrete footings on the tops of two rows of wooden piles nominally spaced 30 in. apart and staggered. The maximum foundation load is estimated to be about 14,000 lb. per linear foot.

The building was commenced last summer, and before its completion had settled and cracked, especially in the rear, so much that it was very badly damaged and in a threatening condition, which grew continually worse. The settlement was very irregular, varying from nothing at some points to 6 in. at other points and producing a great number of cracks, especially where the wall was weakened by the line of windows, which opened up to a width of 2 in. and caused portions of the wall to move away from each other and produced an inclination of 6 or 7 in. out of plumb at the top in one place.

SHORING AND TEMPORARY WALL TIES IN POSITION

The contract for restoring the building to a safe condition was awarded to the Underpinning & Foundation Company, of New York, which at once proceeded to make records of the condition of the building by numerous photographs and bench marks and began operations by shoring the most critical walls on needlebeams and bracing them, where necessary, with spurs. The general principle of repairs was to drive new steel piles under the walls where practicable and just outside the walls in other places, and thus secure enough bearing capacity, combined with the values of the old foundations, to permit the building to be raised, straightened and prevent future settlement.

Ties and Shoring

The walls were in such condition that it was necessary to strengthen them and tie them together at every story, which was accomplished by about twenty longitudinal and ten transverse ties in one building; the other building, being less damaged and having a few permanent steel-rod ties built into it, did not need the temporary ties. Wherever possible the ties were put through window openings and where these were not available holes were cut through the brickwork. Each tie consists of two parts of $\frac{7}{8}$-in. steel guy rope, with the ends connected by a pair of regular bolted clamps. At each end the ties engage the eye of a $1\frac{1}{2}$-in. steel bolt, with a nut at the outer end bearing on a large washer and a distributing timber on the outer surface of the wall. Near the middle of each tie there is a 36-in. turnbuckle, thus giving additional adjustment. At first the ties were adjusted daily to take up the stretch in the rope and pull the walls together as much as possible. This was repeated as often as any opportunity afforded, every two or three days, until after about six weeks the cracks were 50 per cent closed, and it is expected that still more improvement can be effected by the combined use of these ties and of the jackscrews under the needles.

Pits were dug in the cellar down to the tops of the piles and in them and on the surface of the exterior soil large cribs of 12 x 12-in. timber were built up to a height of 16 or 18 ft. and supported sets of two or three 12 or 15-in. I-beams from 20 to 25 ft. long, with which all of the badly cracked walls were needled at short intervals. Each end of each group of I-beams was seated on a single 50-ton screwjack, which was frequently inspected and adjusted to compensate for any settlement and to raise the wall slightly, offsetting some of the displacement.

Use of Steam Hammer

Where the walls were in very bad condition the portions below the street floor, involving in all about 6000 cu. ft. of stonework and 600 ft. of brickwork, were removed down to the tops of the piles and new steel piles 9 in. in diameter, of which there will be at least 50, were driven in staggered rows, one in each face of the wall, to clear the old wooden piles. These piles are made of steel pipes driven in 10-ft. sections with cast-steel conical points. They are driven by a 1000-lb. Goubert steam hammer suspended from the needle beams by a differential hoist. When down to rock they are concreted, their tops capped with $\frac{3}{4}$-in. steel plates and connected by a concrete girder $2\frac{1}{2}$ ft. wide and 3 ft. high continuous through the full length of the wall, and bearing on both the new and old piles. The bottom of this girder is reinforced by four $\frac{3}{4}$-in. longitudinal twisted rods and by transverse rods 6 in. apart on centers. Concrete is made 1:2:4 with Atlas Portland cement.

STEAM HAMMER SUSPENDED FROM NEEDLEBEAMS

Before driving the new piles the muck with which the cellars were backfilled was either shoveled outdoors or was thrown to different parts of the building to expose the original surface of the ground, below which pits were sheeted down about 4 ft. to a depth of 1 ft. below ground-water level, which was about the same as the top of the wooden piles, to insure a proper construction of the new concrete girders. This excavation exposed the tops of the old piles, some of which were found very small, not exceeding 6 in. in diameter, and many of them driven out of line, so that in some cases they were entirely exterior to the old concrete footings. In some cases an attempt was made to drive the wooden piles farther with the steam hammer without causing much additional penetration, but in other cases the old piles moved easily under the hammer.

Jacking Steel Piles

Where the condition of the walls was firm enough to justify it, new steel piles 12 in. in diameter, with open ends, were driven by 60-ton Watson-Stillman hydraulic jacks reacting against the masonry above. As they were driven the interior was cleaned out with a grab bucket, which also successfully removed the small boulders encountered, after which the pipes were filled with concrete and capped. Each pile supports a set of three 12-in. longitudinal I-beams from 4 to 6 ft. long, which are bolted together with separators and wedged and grouted to take bearing against the wall. About fifty of these piles are required for the work as originally contemplated, but it is possible that more of them, as well as of the hammer-driven piles, may be advantageously used before the work is completed. The condition of the buildings as a whole is so critical and the soil so poor that the exact amount and method of underpinning have to be determined largely by the conditions developed as the wall progresses, the intention being to do only what is necessary, but to do that in an efficient manner.

In one place an inner court wall is very badly cracked and settled, while an adjacent cross-wall on similar pile foundations has not settled at all. As the court wall is 4 or 5 in. lower than the original elevation and much of the remainder of the building has settled correspondingly, it will be necessary to lower the unsettled wall a similar amount to restore the integrity of the building. The work was commenced about the first of June and is now being carried on day and night.

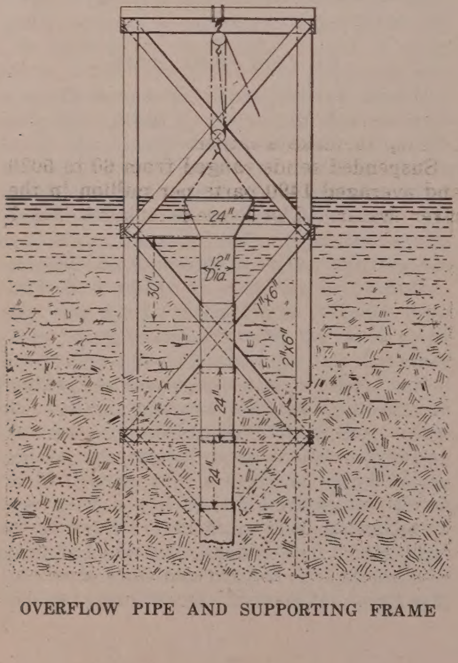
Regulation of Water Levels in Summit Pools

Method Employed in Building San Fernando Earth Dam of Los Angeles Aqueduct

A VERTICAL, adjustable overflow pipe is used to regulate the depth of the summit pool during the construction of the San Fernando Dam of the Los Angeles Aqueduct, as mentioned in the Engineering Record of July 11, page 46. As shown in the accompanying drawing, the pipe is surrounded by a light trestle, carried up with the dam.

The 12-in. overflow pipe has tapered sections 24 in. long and is slip jointed, new sections being inserted as the water rises. At the top there is a loose-fitting, adjustable funnel device inserted in the pipe. This funnel must be weighted to prevent it from floating. With this arrangement the water may be kept at any desired level consistent with the best results. After completion of the dam this pipe, which at its bottom is connected with the permanent outlet conduit through the base of the structure, will be filled with concrete. Since the pipe stands vertically and is embedded in the center of the clay portion of the dam the fact that it makes a smooth line along its outside is of no consequence, as might be the case if the pipe ran transversely through the thickness of the structure.

The dam is being built as part of the Los Angeles Aqueduct under the personal supervision of William Mulholland, chief engineer.



OVERFLOW PIPE AND SUPPORTING FRAME

Unusual Repairs to a Mill

Substitution of Steel Columns for Wood and Buttressing of Outside Wall with Reinforced Concrete Carried Down to Bedrock

By F. A. CAMP
Assistant Engineer for C. A. P. Turner, Minneapolis

THAT a mill building can be entirely rebuilt with the exception of the walls, and these so strengthened as to be more solid than ever, after the wooden posts and girders have become so weakened by dry rot and the walls so shaken by the action of heavy machinery that they are badly cracked and 22 in. out of plumb, and this without stopping the mill or materially reducing its output, is a matter of considerable interest to manufacturers and all owners of buildings. This has recently been done at the Pillsbury A mill in Minneapolis.

About twelve years ago the installation of a large twin waterwheel required the removal of several stone piers, which were replaced with others of cast iron. Shortly after this the head miller became alarmed at the condition of the wooden posts and girders, and at his recommendation a series of four posts lengthwise and crosswise was erected through the center of the mill. They were made up from I-beams and channels in the machine shop of the mill and were erected by mill employees, the conditions not permitting any uniformity of length or even of design, since no outside structural firm could do the work as advantageously as the mill company itself.

As there was some criticism of this work C. A. P. Turner was called in as engineer to examine it. His only criticism was that the work, which was at that time complete up to the third floor, was perhaps a little stronger than necessary.

Replacing Wooden Columns with Steel

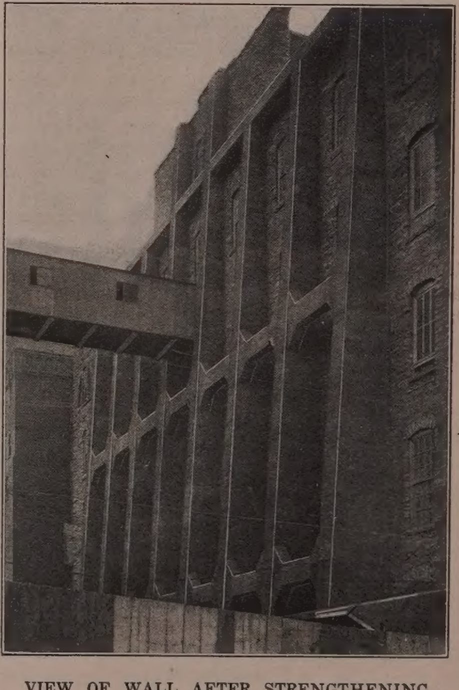
At his suggestion Bethlehem H sections, which had just been placed on the market in large enough sizes, were adopted for the remaining columns. He also advised the use of vertical diagonal braces to stay the mill laterally, but because of the room these would require and the shifting of machinery necessary to accommodate them, this advice was not heeded. The replacement of wooden posts and girders was carried on until the company went into the hands of receivers for reorganization and nothing was done for several years. The building, however, continued to deteriorate rapidly.

After the reorganization the mill was again started. The vibration became alarming and both the north and south walls cracked badly near the corners of the building. The floors seem to have acted as horizontal trusses and to have transmitted the stress to the end walls. Several diagonal braces of 12 x 12-in. timber were put in as a temporary expedient, and in January, 1913, Mr. Turner was called upon to devise something to take the place of the diagonal bracing which he had proposed. Having ascertained that the eastward sag of the building was not due to any settlement of the foundation, he concluded that if the east wall was firmly held in position, steel struts could be run from wall to wall between the joists of each floor to hold the west wall from falling in, this wall having nothing to carry but its own weight and one end of the joists of the west panels. He recommended that monolithic reinforced buttresses be built outside the east wall and that the replacement of wood with steel be extended to include the whole interior construction. This being approved by William DeLabarre, engineer for the mill company, the scheme was adopted and carried out. The interior steelwork was done by mill employees under the direction of William Quayle, head millwright, and the concrete work, both outside and inside, by H. N. Leighton & Company, contractors, the writer superintending it as an assistant to Mr. Turner.

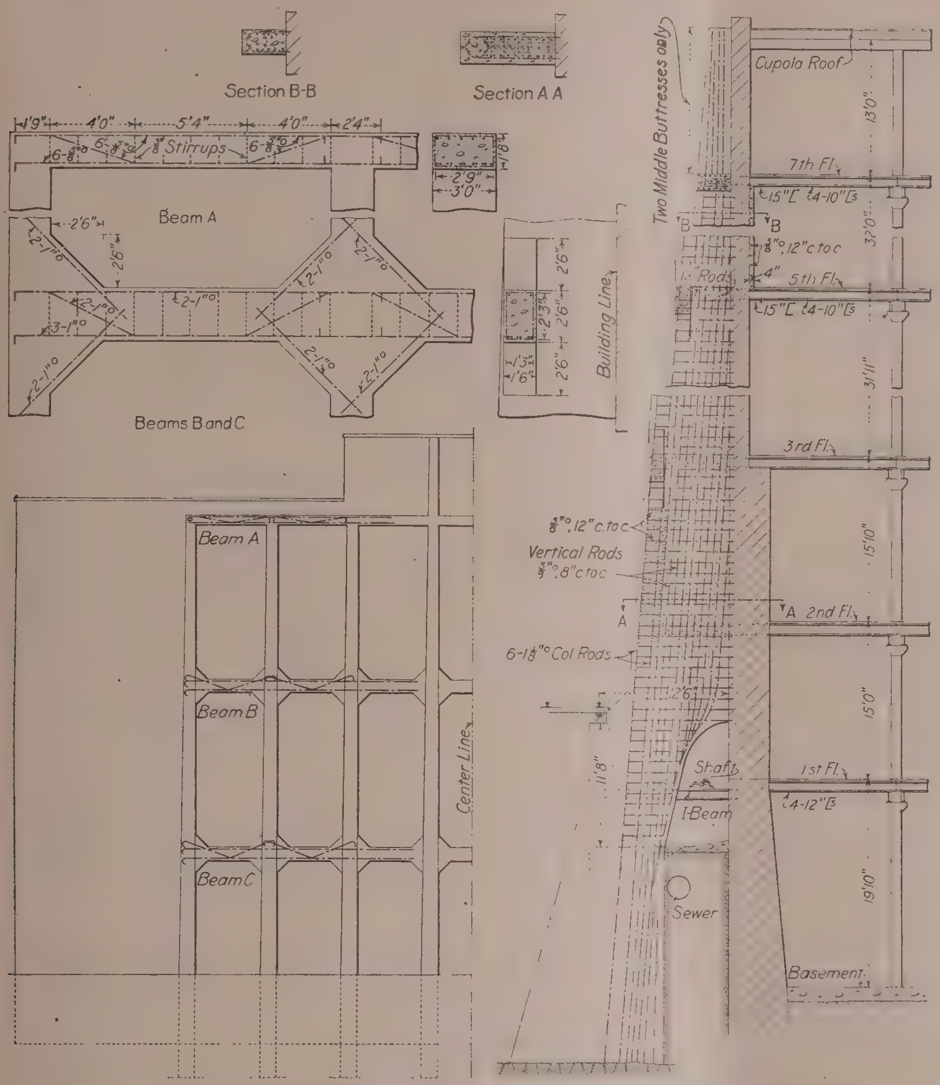
Size and Construction of Mill

The building is about 170 x 108 ft., with six stories and basement in the main part. A cupola about 48 x 108 ft. constitutes a seventh floor. The total height from basement floor to cupola roof is 127 ft. 6 in. There are sixty columns arranged in six rows of ten each, dividing the floor space into panels about 15 ft. 6 in. square and extending from basement floor to main roof. They are tied together by two lines of 12-in. I-beams lengthwise and either two lines of 10-in. I-beams or four lines of 10-in. channels crosswise at each floor. These extend to the wall and are anchored thereto. The 12-in. longitudinal I-beams are supported on brackets riveted to the columns. They carry the floor joists. The transverse 10-in. I-beams and channels carry no loads. The former are fastened to the flanges of the vertical channels of the built-up columns with 3 x 5-in. angles riveted to their webs. At the H-section columns the 10-in. channels are placed one on each side of the flanges, with beveled washers to fit the flanges interposed on one side, and rivets are driven through both channels and the flanges. Under all floors except the third, where there is an offset in the walls, a line of 15-in. channels is riveted to the under side of the transverse struts about 4 in. from the wall.

Starting on the offset, a reinforced-concrete lining to the east and west walls,



VIEW OF WALL AFTER STRENGTHENING



GENERAL ARRANGEMENT AND DETAILS OF REINFORCED-CONCRETE COUNTERFORTS

varying from 4 to 7 in. in thickness on account of the irregularity, was made up to the roof. This was extended a short distance on the north and south walls where they were badly cracked. The old plastering was entirely removed and the joints were dug out as deep as possible with narrow chisels. New window frames with openings somewhat smaller than the old ones were provided and the concrete was continued around the jamb corners to the faces of these frames. The jambs were reinforced with wire mesh, which was turned around the corners and extended about a foot parallel to the walls. The horizontal reinforcement was increased over the openings to such an extent that the amount of steel used was practically the same as if the general reinforcement had continued across the openings. This concrete backing binds the cracked walls into a monolithic mass stronger than the walls were when new.

In the original timber construction the outer ends of the joists were carried on trimmers over some of the windows. These were removed and the joists allowed to rest on the 15-in. channels previously mentioned, which in this story were supported by the addition to the concrete lining of a 15-in. projection similar in section and reinforcement to half of a "mushroom" column head.

From the engineering standpoint, however, the main feature of the whole scheme, without which all the above-described work would be useless, is the eight monolithic reinforced-concrete flying buttresses built from bedrock, 34 ft. below the tracks, to the roof. These, with the exception of the two rows nearest the end walls, are located in the line of the transverse rows of columns, the transverse I-beams and channels extending through the east wall and being anchored into them. The two middle ones are 28 x 30 in. at the main roof level and 3 x 9 ft. at a point 2 ft. above the tracks. The others are somewhat larger at the top, as the faces are in a straight line and the wall is bulged out in the middle.

ARCHING OF BUTTRESSES

Because of a line of shafting and other machinery in an area, and sewer and water pipes underneath, it was not possible to place these buttresses against the walls any further down than about 2 ft. below the tracks. At this height they spring from the wall in a parabolic curve which brings the inside 6 ft. from the wall in a drop of 12 ft. From this height down to bedrock the inner face is vertical. About 2 ft. above the tracks the batter of the outer face is increased to 1 to 1 for 3 ft. in height to meet a timber track stringer and leave the necessary clearance at the height of a car floor. Below this the batter is 3½ in per foot. On the bedrock the deepest buttress is 3 x 15 ft. and its outer face is 21 ft. from the face of the wall. The buttresses are connected together by concrete beams 18 x 30 in. at the second and fourth floor levels, these being close to the outer face and joined to the buttresses by brackets above and below, making a total height of 6 ft. 6 in. on the side faces. At the main roof level is a beam 20 in. high and from 30 to 36 in. wide, its face being straight and its back fitting the wall.

The buttresses are reinforced vertically with rounds in a spiral placed near the outer face and inclining with it. Near each side face of the buttresses are horizontal and vertical rods as shown. It was intended that these horizontal rods should extend into the stone wall, but as it was found difficult to maintain even spaces they were bent up or down at the ends and an equal number of ⅜-in rounds 24 in. long were driven as far as possible into the joints of the stonework. The joints were all cut out as deep as possible with narrow chisels, so that the concrete might flow into them, thus obtaining a firm bond, the absolute rigidity of the joints between buttresses and wall being the vital feature of the whole scheme. In the writer's opinion these buttresses if built of ordinary masonry could not have been made sufficiently rigid to accomplish the desired end, and in the opinion of some engineers even the monolithic ones would not do it; but "when this work was completed," to quote from a letter from Superintendent Lehman, "the vibration of the mill practically stopped. The total cost was approximately $150,000, but the mill was not shut down for a single day." The cost of the interior steelwork was about $120,000 and of the concrete work about $30,000.

Results of Water-Clarification Plant at St. Louis

Figures Taken from the Annual Report of E. E. Wall, Water Commissioner of the City

LIME and iron to the extent of 5.63 and 2.49 grains per gallon were used last year to clarify the water of the Mississippi River at St. Louis, according to the last annual report of the water commissioner, E. E. Wall. The maximum and minimum figures for lime were 7.5 and 4.25 grains respectively, while for the iron the corresponding quantities were 4.25 grains and 1 grain. The average costs were $1.66 for lime and $1.73 for iron. A small amount of chlorine was used during the latter part of the year. Alkalinity was reduced from 133 to 50 parts per million and non-carbonate hardness increased from 43 to 57. Total hardness was reduced from 186 to 107 parts per million, corresponding to a saving of 80 cents per 1000 gal. in the cost of soap to make a lather.

Suspended solids ranged from 60 to 5020 and averaged 1490 parts per million in the river water. The average effluent had 7 parts per million left. Bacteria on agar at 37 deg. C. averaged 7200 and 54 per cu. cm in the river and treated water. B. coli showed presumptively on 235 of the 263 days tested on the river water in 0.01-cu. cm samples.

Irregularities in flow of chemical ranging from 45 to 109 per cent of the estimated were attributed to caking of the iron, inaccuracy in adjusting the orifice and variation in the size of particles, with consequent variation in voids. The error is considerable and is fundamental, according to W. F. Montfort, chemist, having its source in the principle of volume measurement through an orifice; it can be obviated, he states, by the use of Richardson's automatic scales used in weighing lime.

Clogged 6-in. cast-iron pipe lines carrying lime for the coagulant house to the delivery well were cleaned at a cost of 32 cents per linear foot by the National Water Main Cleaning Company. The milk of lime is pumped at a temperature of 78 deg. Fahr. and carries 250,000 parts per million of lime. The final stoppage occurred at a rise and was repaired by the department, which took out the sections and heated and cleaned them.

Seven times the settling basins at the Chain of Rocks have been cleaned during the year at a cost of 1.42 cents per cubic yard for 150,000 cu. yd. removed from four basins. These costs are for labor and teams only. Water lost in drawing down the basins was 200,000,000 gal. and that used during cleaning 130,000,000 gal. Mud valves were opened 360 hr. for flushing out loose mud. The efficacy of flushing was shown when construction of the connections for the filter-plant drain necessitated omitting the operation for three weeks. Soft mud so thick that it would not flow filled the basin within 5 ft. of the coping.

Sanitary Experts Answer Joint Commission's Questions

Essence of Testimony on Methods of Remedying Conditions of Pollution in Great Lakes Boundary Waters

AFTER the International Joint Commission had completed its sanitary survey of the Great Lakes, thereby finishing the first stage of its program by determining the extent and causes of the pollution of the boundary waters between the United States and Canada, work was immediately begun on the second of its missions—to suggest means of remedying such troubles as were found to exist. The procedure has been substantially in accordance with the recommendations which Dr. Allan J. McLaughlin made some time ago. These were:

1. Secure the opinion of the most eminent sanitary engineers by submitting a list of questions. These opinions would form a sound basis for formulating a general policy and possibly some general minimum requirements.
2. Hold public hearings at which municipalities could present their cases, including expert engineering testimony, plans and projects.
3. Employ expert sanitary engineers for the formulation of special standards for each waterway in excess of general minimum requirements, depending on local conditions and necessities.

CONFERENCE IN NEW YORK

Recently the commission held a conference in New York with the following sanitary engineers: George W. Fuller, George C. Whipple and Prof. Earle B. Phelps, for the United States, and F. A. Dallyn, W. S. Lea and T. J. Lafreniere, representing Canada. A list of questions was submitted to this board and the answers are appended. The testimony on the main facts of pollution, its results and remedies, was in almost unanimous agreement. Mr. Dallyn, however, objected to several points and asked for the elimination of paragraphs 5, 7 and 11 in their entirety. His name is not signed to the following report embodying the answers, which was made June 27:

1. Speaking generally, water supplies taken from streams and lakes which receive the drainage of agricultural and grazing lands, rural communities, and unsewered towns are unsafe for use without purification, but are safe for use if purified.

2. Water supplies taken from streams and lakes into which the sewage of cities and towns is directly discharged are safe for use after purification, provided the load upon the purifying mechanism is not too great and that a sufficient factor of safety is maintained, and, further, provided the plant is properly operated.

3. As, in general, the boundary waters in their natural state are relatively clear and contain but little organic matter, the best index of pollution now available for the purpose of ascertaining whether a water-purification plant is overloaded is the number of B. coli per 100 cu. cm. of water expressed as an annual average and determined from a considerable number of confirmatory tests regularly made throughout the year.

SAFE LOADING OF PURIFICATION PLANT

4. While present information does not permit a definite limit of safe loading of a water-purification plant to be established, it is our judgment that this limit is exceeded if the annual average number of B. coli in the water delivered to the plant is higher than about 500 per 100 cu. cm. or if in 0.1 cu. cm. samples of the water B. coli is found 50 per cent of the time. With such a limit the number of B. coli would be less than the figure given during a part of the year and would be exceeded during some periods.

5. In waterways where some pollution is inevitable and where the ratio of the volume of water to the volume of sewage is so large that no local nuisance can result, it is our judgment that the method of sewage disposal by dilution represents a natural resource and that the utilization of this resource is justifiable for economic reasons, provided that an unreasonable burden or responsibility is not placed upon any water-purification plant and that no menace to the public health is occasioned thereby.

6. While realizing that in certain cases the discharge of crude sewage into the boundary waters may be without danger, it is our judgment that effective sanitary administration requires the adoption of the general policy that no untreated sewage from cities or towns shall be discharged into the boundary waters.

TREATMENT TO SUIT LOCAL CONDITIONS

7. The nature of the sewage treatment required should vary according to the local conditions, each community being permitted to take advantage of its situation with respect to local conditions and its remoteness from other communities, with the intent that the cost of sewage treatment may be kept reasonably low.

8. In general, the simplest allowable method of sewage treatment, such as would be suitable for small communities remote from other communities, should be the removal of the larger suspended solids by screening through a ¼-in. mesh or by sedimentation.

9. In general, no more elaborate method of sewage treatment should be required than the removal of the suspended solids by fine screening or by sedimentation, or both, followed by chemical disinfection or sterilization of the clarified sewage. Except in the case of some of the smaller streams on the boundary, it is our judgment that such oxidizing processes as intermittent sand filtration and treatment by sprinkling filters, contact beds, and the like, are unnecessary, inasmuch as ample dilution in the lakes and large streams will provide sufficient oxygen for the ultimate destruction of the organic matter.

10. Disinfection or sterilization of the sewage of a community should be required wherever there is danger of the boundary waters being so polluted that the load on any water-purification plant becomes greater than the limit above mentioned.

11. It is our opinion that, in general, protection of public water supplies is more economically secured by water purification at the intake than by sewage purification at the sewer outlet, but that under some conditions both water purification and sewage treatment may be necessary.

BACTERIOLOGICAL TESTS

12. The bacteriological tests which have been made in large numbers under the direction of the International Joint Commission indicate that in some places the pollution of the boundary waters is such as to be a general menace to the public health should the water be used without purification as sources of public water supply or should they be used for drinking purposes by persons traveling in boats.

13. It is our judgment that the drinking water used on vessels traversing boundary waters should not be taken indiscriminately from the waters traversed, unless subjected to adequate purification, but should be obtained preferably from safe sources of supply at the terminals.

14. While recognizing that the direct discharge of fecal matter from boats into the boundary waters may often be without danger, yet in the interest of effective sanitary administration it is our judgment that the indiscriminate discharge of unsterilized fecal matter from vessels into the boundary waters should not be permitted.

Concrete-Lined Steel Pipe

Built in Baltimore Recently to Deliver Water from Loch Raven Reservoir

A 10-FOOT concrete-lined steel pipe recently built in Baltimore to deliver water from the Loch Raven reservoir to the Loch Raven-Montebello tunnel at a point just below its gate house is described by Ezra B. Whitman, president of the Water Board, in the monthly journal of the Engineers' Club of Baltimore. It was estimated that this pipe could be constructed and that the relining of 1 mi. of the old 12-ft. tunnel could be done for an estimated cost of about $300,000, as compared with the estimated cost of $480,000 for a new 12-ft. tunel 6340 ft. long joining the old tunnel 1 mi. below its upper end. The pipe is made in alternate inside and outside telescoping lengths 15 ft. long and of 10-ft. minimum inside diameter. Changes in grade and alignment are made by beveling the ends of the section. The pipe is made of soft open-hearth rivet steel intended to have sufficient ductility to stretch rather than crack in case of unusual strain from water hammer or settlement. The metal is 7-16 in. thick, although a thickness of less than ¼ in. would theoretically suffice for the stress developed. The pipe is supported on concrete cradles intermediate between the riveted joints and having a radius slightly greater than that of the pipe surface.

Problem in Railway Curves

To Pass an Interchange Track of Maximum Uniform Radius around a Limiting Corner, where the Two Frog Angles Differ

By F. C. SNOW
Assistant Professor of Civil Engineering, Montana State College, Bozeman

INTERCHANGE tracks between two railroads crossing at grade are likely to be located on as long a radius as is compatible with right-of-way or building corners. Such a track will properly have a short tangent between heel of frog and point of curvature and the standard frog angles for the two railroads may differ. Thus there arises the problem shown in the accompanying figure, in which the two tracks AQ and BQ intersect at the point Q with the angle θ, the frogs at A and B are of the respective angles α and β and a regular curve with short tangents of length e at each end is to pass through the point P, located from the intersection by the distances p and q. Required the radius R and the distances x and y.

First solving the figure $LCAOPL$ by latitudes and departures we have the following, in which φ and R are unknown:

Course	Bearing	Distance	N	S	E	W
LC	N	R	R			
CA	W	e				e
AO	$N(90-\alpha)E$	$x-p$	$(x-p)\sin\alpha$		$(x-p)\cos\alpha$	
OP	$S\alpha E$	q		$q\cos\alpha$	$q\sin\alpha$	
PL	$S\phi W$	R		$R\cos\phi$		$R\sin\phi$

Since $\Sigma N = \Sigma S$ and $\Sigma E = \Sigma W$,

$$R \cos \varphi = (x - p) \sin \alpha + R - q \cos \alpha \quad (1)$$

and

$$R \sin \varphi = (x - p) \cos \alpha - e + q \sin \alpha \quad (2)$$

Squaring equations (1) and (2) and adding,

$$R^2 = [(x-p) \sin \alpha + R - q \cos \alpha]^2 + [(x-p) \cos \alpha - e + q \sin \alpha]^2$$

or

$$R = \left\{ e[q \sin \alpha + (x-p) \cos \alpha] - \tfrac{1}{2} [(x-p)^2 + q^2 + e^2] \right\} \div [(x-p) \sin \alpha - q \cos \alpha] \quad (3)$$

This gives R in terms of x, both being unknown.

Now solving the figure $AQBFA$ by the same method we have the following:

Course	Bearing	Distance	N	S	E	W
AQ	N	x	x			
QB	$N\theta E$	y	$y\cos\theta$		$y\sin\theta$	
BF	$S(\theta-\beta)W$	$R\tan\frac{1}{2}\Delta + e$		$(R\tan\frac{1}{2}\Delta+e)\cos(\theta-\beta)$		$(R\tan\frac{1}{2}\Delta+e)\sin(\theta-\beta)$
FA	$S\alpha W$	$R\tan\frac{1}{2}\Delta + e$		$(R\tan\frac{1}{2}\Delta+e)\cos\alpha$		$(R\tan\frac{1}{2}\Delta+e)\sin\alpha$

Hence

$$y \sin \theta = (R \tan \tfrac{1}{2} \Delta + e) [\sin (\theta - \beta) + \sin \alpha] \quad (4)$$

and

$$y \cos \theta = (R \tan \tfrac{1}{2} \Delta + e) [\cos (\theta - \beta) + \cos \alpha] - x \quad (5)$$

Eliminating y from equations (4) and (5),

$$\frac{(R \tan \frac{1}{2} \Delta + e) [\sin (\theta - \beta) + \sin \alpha]}{\sin \theta} = \frac{(R \tan \frac{1}{2} \Delta + e) [\cos (\theta - \beta) + \cos \alpha] - x}{\cos \theta}$$

from which

$$\frac{x}{\cos \theta} = \frac{(R \tan \frac{1}{2} \Delta + e) [\cos (\theta - \beta) + \cos \alpha]}{\cos \theta} - \frac{(R \tan \frac{1}{2} \Delta + e) [\sin (\theta - \beta) + \sin \alpha]}{\sin \theta}$$

and

$$x = (R \tan \tfrac{1}{2} \Delta + e) \left\{ \cos (\theta - \beta) + \cos \alpha - [\sin (\theta - \beta) + \sin \alpha] \cot \theta \right\} \quad (6)$$

From equation (4),

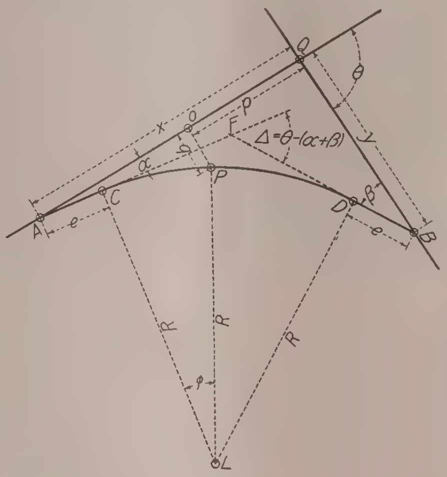
$$y = \frac{(R \tan \frac{1}{2} \Delta + e) [\sin (\theta - \beta) + \sin \alpha]}{\sin \theta} \quad (7)$$

Also

$$\Delta = \theta - (\alpha + \beta)$$

To work the problem, solve equation (6) for x in terms of R and equation (3) for R. Correct R for the curve having the nearest minute degree, then figure the final value of x from equation (6) and y from equation (7).

As an example, suppose the two tracks intersect at an angle of 40 deg., the frogs at A and B are to be respectively a No. 10 and a No. 6, the short tangents at the heels of the frogs are to be 10 ft. and the limiting point P is 400 ft. from Q on the line AQ and 50 ft. at right angles thereto. Then $\theta = 40$ deg., $\alpha = 5$ deg. 43 min., $\beta = 9$ deg. 31 min., $e = 10$ ft., $p = 400$ ft. and $q = 50$ ft. From equation (6)



$$x = (R \text{ tan } 12 \text{ deg. } 24 \text{ min. } + 10) \{\cos 30 \text{ deg. } 29 \text{ min. } + \cos 5 \text{ deg. } 43 \text{ min. } - [\sin 30 \text{ deg. } 29 \text{ min. } + \sin 5 \text{ deg. } 43 \text{ min.}] \cot 40 \text{ deg.} \} = 0.2493\, R + 11.335$$

or, since R will have to be adjusted

$$x = 0.25\, R + 11.34$$

From equation (3)

$$R = \{10[50 \times 0.09961 + (x - 400)\, 0.99503] - \tfrac{1}{2} [x^2 - 800\, x + 160{,}000 + 2500 + 100]\} \div [(x - 400)\, 0.09961 - 50 \times 0.99503]$$

$$= \frac{409.95\, x - 0.5\, x^2 - 85{,}230.3}{0.1\, x - 89.6}$$

Substituting the value of x found above,

$$R = [409.95\, (0.25\, R + 11.34) - 0.5\, (0.25\, R + 11.34)^2 - 85{,}230.3] \div [0.1\, (0.25\, R + 11.34) - 89.6]$$

Solving the quadratic and using the proper root, $R = 2853$. The nearest minute degree corresponding to this value is 2 deg., for which

$$R = 2864.93$$

Substituting this value of R in the equation derived for x,

$$x = 725.57$$

and from equation (7)

$$y = 604.14$$

In applying the formulæ it will be seen that the points N and T are the actual frog points and that the lines in the figure are gage lines—of the inner rails of the intersecting tracks and of the outer rail of the curved track. In order to clear the corner of a building or a right-of-way angle at the point P, therefore, it will be necessary to make an approximate allowance of a few feet in determining p and q, so that with the outer rail passing through P equipment on the track will not foul the limiting corner.

The Auburn Water Case

Sanitary Lawsuit Involving the Carrying of Pollution by Wind and Water Movements

A VERDICT was rendered at Auburn, N. Y., on April 1 in the case of the People against the Board of Education of the Union Free School of Moravia which, according to the monthly bulletin of the New York State Department of Health, promises to set an important precedent in connection with the problems of sewage disposal and water pollution.

The Moravia high school discharges the sewage of some 300 pupils by a 4-in. sewer upon the bank of Mill Creek, a small stream flowing by way of Owasco Inlet into the southern end of Owasco Lake. At the northern end of the lake the City of Auburn (population about 35,000) uses the lake for its public water supply. An indictment was found in 1911 against the trustees of the school for maintaining a public nuisance. Three counts charged them with "offending public decency," "annoying, injuring and endangering the comfort, repose, health or safety" of a considerable number of persons and rendering "a considerable number of persons insecure in life or the use of property."

The case was first tried in the spring of 1913 and resulted in a disagreement, the jury standing ten for conviction and two for acquittal. The second trial was begun in March of the present year, District Attorney Albert H. Clark and John M. Brainard, the attorney of the Auburn Water Board, being counsel of record for the prosecution and County Judge Hull Greenfield and James A. Wright acting as chief counsel for the defense. Prof. W. P. Mason of Rensselaer Polytechnic Institute, Dr. F. M. Meader of Syracuse, Nicholas S. Hill, Jr., and Prof. C.-E. A. Winslow of New York City were the principal expert witnesses for the people.

FACTS IN THE CASE

It was shown that considerable masses of fecal matter accumulated on the bank of Mill Creek at the mouth of the sewer to be washed down into Lake Owasco, about 4 miles distant, at times of rain. It was shown that in the autumn of 1907 the village of Moravia suffered from an epidemic of some twenty cases of typhoid fever and that at least three cases occurred among pupils of the high school, two of whom returned as convalescents in the spring. During the winter the lake was, of course, frozen, but in the middle of March, 1908, there was a heavy thaw, the ice broke up and the level of the lake rose rapidly. At the same time great increase was apparent in the bacterial counts in the Auburn water. In April a sharp epidemic of typhoid broke out in Auburn including thirty-four cases in two months. The 1908 outbreak was distributed very evenly over the city and had all the characteristics of a water epidemic. Since no other cases of typhoid were known to have polluted the waters tributary to Owasco Lake the chain of evidence pointing to Moravia village as the source of the Auburn typhoid was a strong one.

Sanitarians familiar with the protective effect of storage would naturally however question the possibility of the transmission of virulent typhoid germs down a lake 10 mi. long and 1 mi. wide with an almost negligible current. This difficulty was met by a remarkable study of wind currents carried out by J. Walter Ackerman, engineer of the Auburn water board. Mr. Ackerman, by float measurements, worked out the relation between wind movement and surface water movement at different wind velocities and showed that with south winds, which prevailed in the spring of 1908 for considerable periods, surface water could easily be carried from the head to the foot of the lake in two or three days.

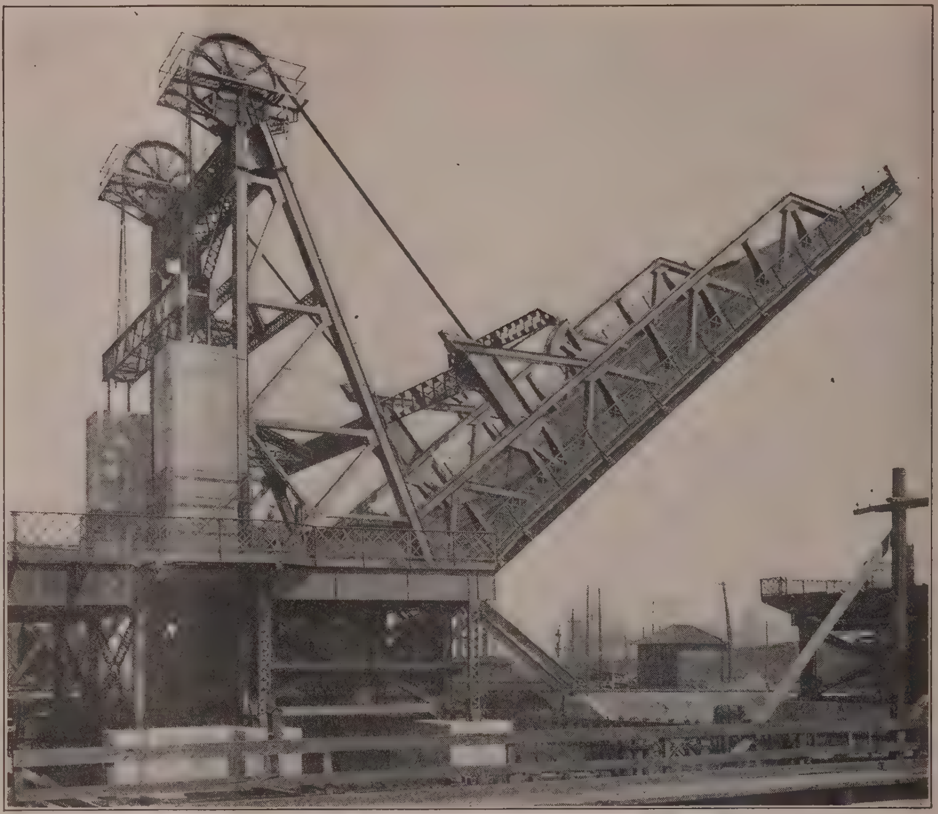
The defense denied the possibility of such transmission on general grounds and pointed to nearer sources of pollution than Owasco inlet and called attention to many other points of serious pollution than the high school sewer along Mill Creek.

Arguments and Results

District Attorney Clark in his summing up emphasized the difference between ordinary rural pollution which constitutes a proper use of water courses and which must be remedied at the cost of a water company or corporation using the water for drinking and an unusual, extreme case like that of the high school which may properly be held to constitute a nuisance.

The jury found for the defendant on the first two counts but convicted on the third count of rendering "a considerable number of persons insecure in life or the use of property."

The case is of interest from the scientific standpoint on account of the light it throws on the effect of wind currents and the consequent danger of long distance water pollution, and from the administrative standpoint on account of the comparatively untried method it suggests for dealing with aggravated conditions of water pollution.



BASCULE SPAN OPERATED BY HOISTING ENGINE

Hamburg Turnpike Lift Bridge

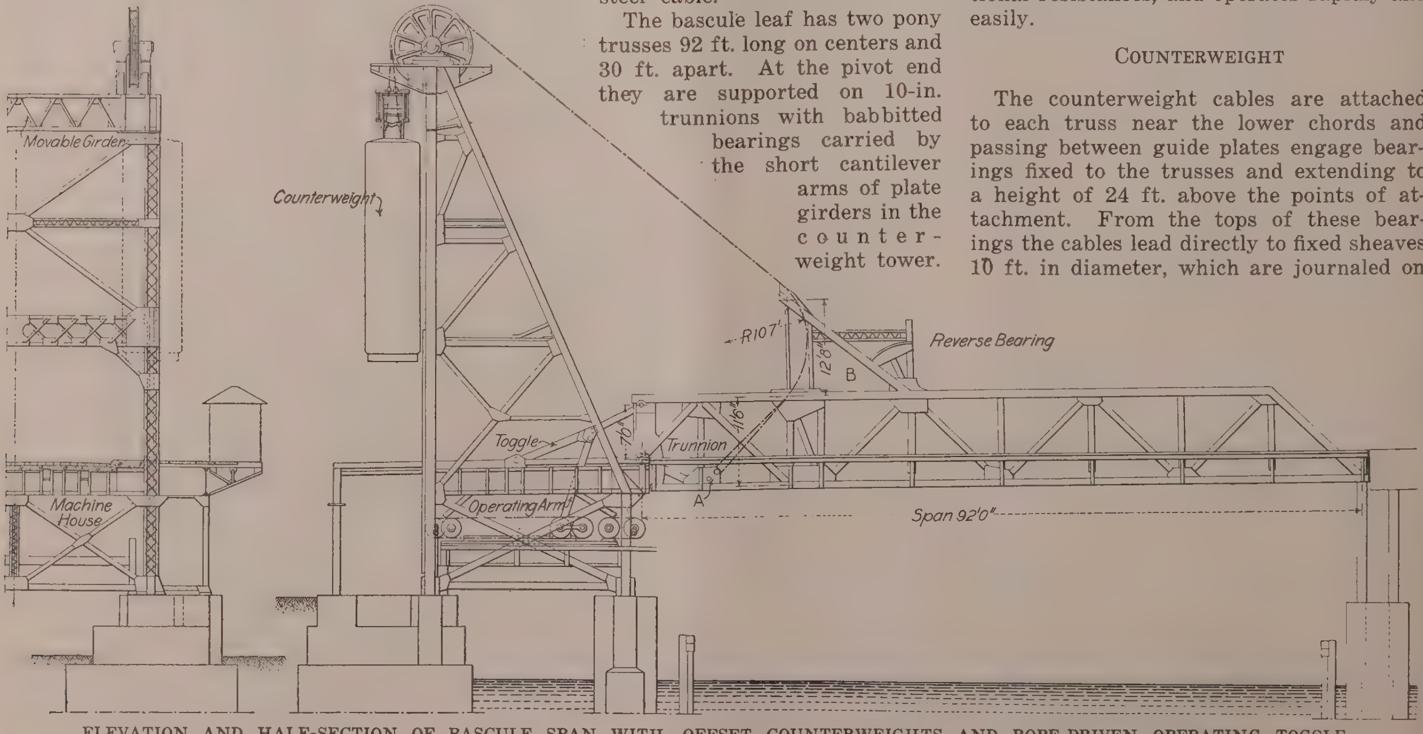
Cable-Operated Toggle Power Highway and Electric-Car Bascule Span over the Ship Canal of the Buffalo Creek Railroad

The Hamburg turnpike viaduct is part of the grade-crossing elimination work in Buffalo, N. Y., and carries a driveway, two electric-car tracks and two sidewalks over the yards and ship canal of the Buffalo Creek Railroad. The canal span has a single bascule leaf with a counterweight exactly balancing it in every position and a rapid-action operating toggle driven by a steel cable.

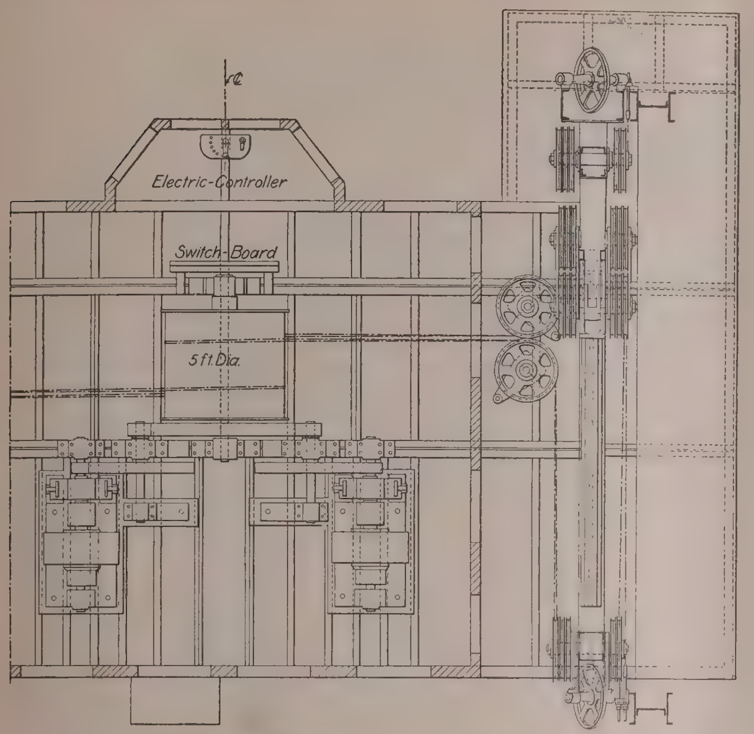
The bascule leaf has two pony trusses 92 ft. long on centers and 30 ft. apart. At the pivot end they are supported on 10-in. trunnions with babbitted bearings carried by the short cantilever arms of plate girders in the counterweight tower. At the opposite end they have seats on a steel bent about 16 ft. high above the top of the masonry pier. The trusses are revolved through a vertical arc of about 90 deg. by the operation of toggles attached to them at points 7 ft. vertically above the trunnion pins. The dead load of the span is fully and accurately counterbalanced, so that the toggle has only to overcome wind and frictional resistances, and operates rapidly and easily.

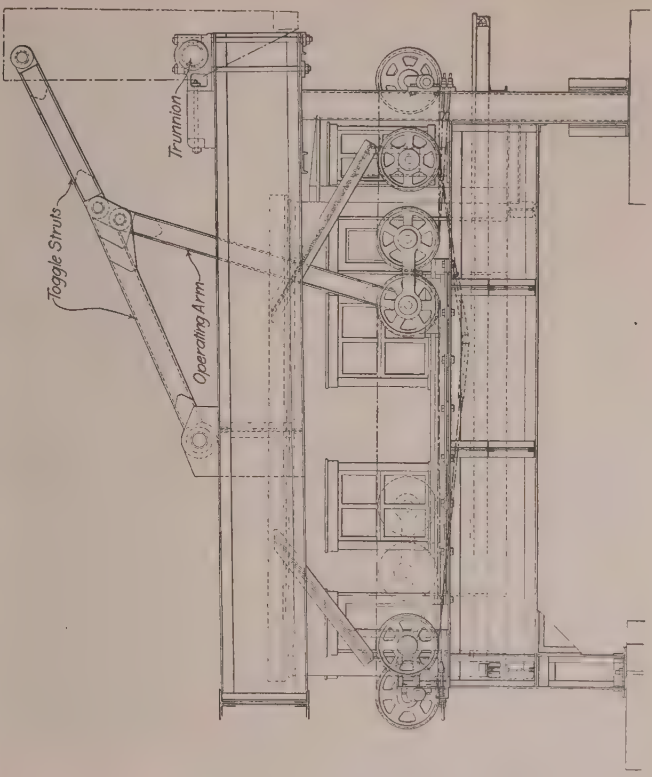
Counterweight

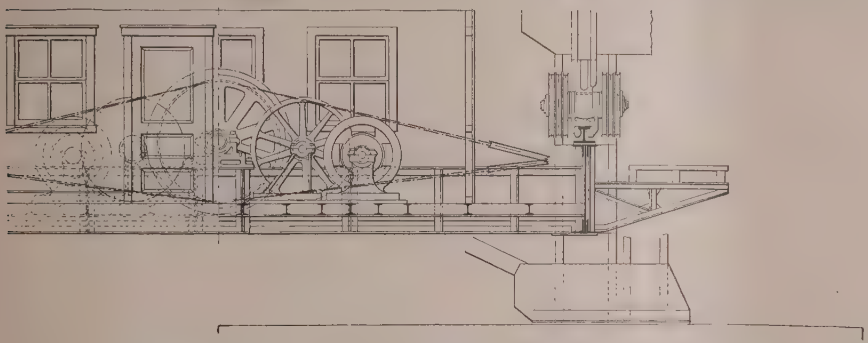
The counterweight cables are attached to each truss near the lower chords and passing between guide plates engage bearings fixed to the trusses and extending to a height of 24 ft. above the points of attachment. From the tops of these bearings the cables lead directly to fixed sheaves 10 ft. in diameter, which are journaled on



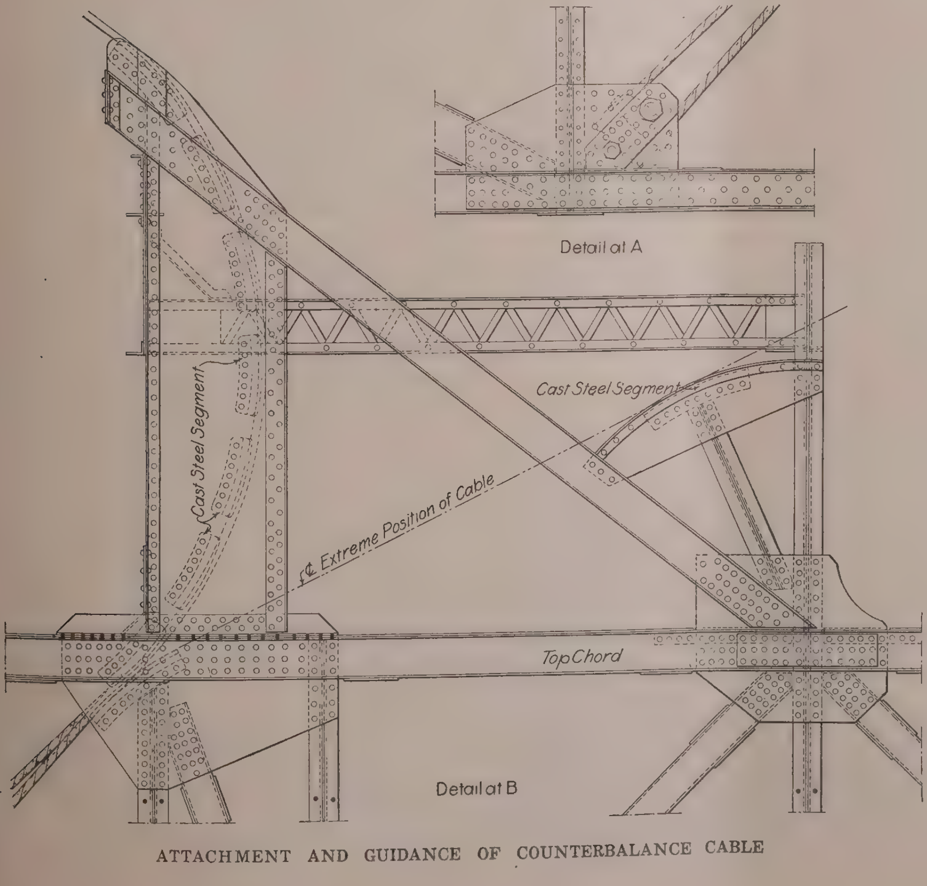
ELEVATION AND HALF-SECTION OF BASCULE SPAN WITH OFFSET COUNTERWEIGHTS AND ROPE-DRIVEN OPERATING TOGGLE







TOGGLE, OPERATING CABLE, HOISTING ENGINE AND MOTORS



Detail at A

Detail at B

ATTACHMENT AND GUIDANCE OF COUNTERBALANCE CABLE

top of the 70-ft. tower. On the shore side of sheaves the cables are attached to a double horizontal transverse movable girder, which at both ends cantilevers beyond the sheaves to receive at the extremities the connections to the counterweight, which are thus offset, outside the planes of the trusses, far enough to give full clearance for roadway traffic.

As the bascule trusses revolve upward and their centers of gravity approach the verticals through the trunnions, the curved bearings for the counterweight cables also revolve, and, being eccentric with the trunnions, also approach the verticals through the latter. The cables become tangent to the curved bearings at a constantly diminishing lever arm from the trunnion, which is accurately calculated to make the moment of the constant pull automatically equal to the changing moment of the span, and thus neither accelerate nor retard its movement.

CABLE BEARING

As the bridge opens the cable bearing changes from the upper to the lower end of its curved support, and as the bridge closes the reverse takes place. The cables being thus always under uniform tension, and the counterweights having a regular vertical movement, are calculated to promote the steady operation of the bridge. The framework which carries the cable bearing above the top chords of the trusses is provided with a reverse bearing to engage the cable, and serve, if necessary, as a buffer at the extreme upward position of the trusses when their gravity moment is very small.

The counterweights, about 7 ft. square and 29 ft. long, are of concrete with a steel framework. They have a combined weight of 420,000 lb. and a vertical movement of 28.8 ft., the lower ends descending about 14 ft. below grade. Steel-frame railings 8 ft. high, covered with small mesh wire netting, inclose the counterweight wells to protect them and prevent the possibility of their becoming obstructed, or of any person being injured by the counterweight.

Each truss is pivoted on a bottom-chord trunnion and is provided with a toggle connected to the shore end of the top chord. The toggle consists of two riveted struts which, when the bridge is closed, lie in a nearly straight line inclined about 25 deg. to the horizontal and form a continuous member about 17 ft. long, the lower end of which is securely anchored by a fixed pin to a plate girder in the base of the tower. One toggle strut is pivoted to the operating arm, the opposite end of which has a sliding bearing on a horizontal track parallel with the bridge axis.

Two sets of sheaves are connected to the bottom of the operating arm, each of them engaging the bights of horizontal wire ropes that are anchored at opposite ends of the longitudinal tower girders supporting the mechanism. The free end of each set of ropes is led to and wound on a large drum at the center of the tower driven by two 30-hp Otis electric motors.

Both sets of ropes are operated simultaneously and equally by the drum, and when the latter revolves in one direction it pulls the foot of the operating arms toward the shore, depresses the toggles and opens the bridge, one set of the ropes overhauling as the other set is wound up. Reversing the drum reverses all of the rope motions, and the operating arms are pulled back toward the river, elevating the toggles and closing the bridge. By this arrangement the bridge slows down automatically and seats itself gently and firmly, though the motors may run at full speed; thus the seating does not depend on the section of the brakes or the skill of the operator, and end locks are unnecessary.

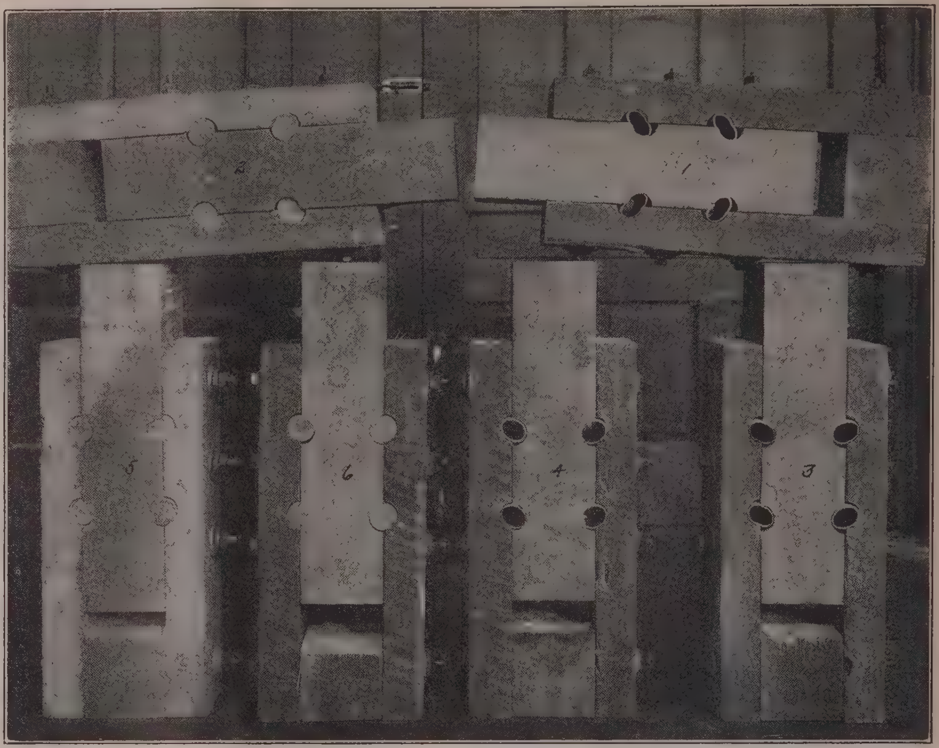
Automatic Stops

Automatic stops are provided to cut off the current after the bridge is seated, but in case these should be neglected no harm can arise, since the continued operation of the driving drum after the bridge is closed merely continues to actuate the toggle, tending to reverse the motion of the bridge without injury.

This system involves no difficult adjustment, complicated mechanism, or parts subject to derangement and frequent renewal. The curved bearings for the counterweight cables are cast-steel blocks riveted between vertical steel guide plates after the exact weight and center of gravity of the bascule span had been determined from the finished detailed drawings and the position of each segmental piece accurately fixed. All of the principal bearings have babbitted surfaces in phosphor-bronze journals or bushings and are proportioned for 700 lb. per square inch of projected area under maximum load.

The bridge was designed for the Buffalo Engineering Company by Thomas E. Brown, consulting engineer, of New York, and approved by the late C. W. Buckholz and by F. G. Ward, commissioner of public works. The United Construction Company, of Albany, N. Y., was the general contractor. The steel was fabricated by the American Bridge Company, and the hoisting engine, motors and electric equipment were furnished by the Otis Elevator Company.

FORESTATION of the land around the Wachusett Reservoir of the city of Boston has entailed planting of 125,000 white-pine seedlings.



TEST OF JOINTS 1A, 2A, 3A, 4A, 5A AND 6A

Resistance of Timber Joints

Compression Tests of Splices Made with Wood and Iron Keys for the Panama-Pacific Exposition Buildings at San Francisco

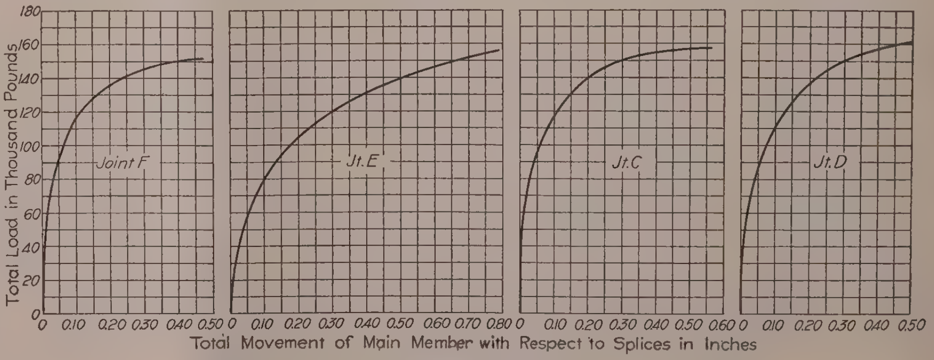
By ARTHUR C. ALVAREZ
Assistant Professor of Civil Engineering, University of California

IN connection with problems arising in the design of structural timber details for Machinery Hall in San Francisco (see Engineering Record of Aug. 30, 1913, page 237) investigations have been made for the Division of Works of the Panama-Pacific International Exposition Company on the strength of timber splice joints reinforced with round pins of various kinds of materials, such as hard woods, solid iron and gaspipes. These investigations were made by the writer at the University of California in the civil engineering testing laboratory, of which Charles Derleth, Jr., professor of civil engineering and dean, is director.

All joints were designed under the direction of H. D. Dewell, chief structural draftsman of the Division of Works. All joints were fabricated in the field by W. W. Anderson & Company, the contractors for Machinery Hall.

Douglas fir was used for the main members and splices of all joints. All pins were 2 in. in external diameter and extended the full depth of the joint—from the front to the rear face. All timber was of good sound grade. On an average there were between eleven and fourteen annular rings per radial inch. However, many of the splices of joints given in Table 2 had only four rings per radial inch. The efficiency of such coarse-grained timber in a joint is much less than that of stronger, finer-grained material. It is softer, so that washers pressing perpendicular to its grain sink in more easily. It is weaker to resist any bending coming on splices in planes perpendicular to the longitudinal axis of the entire specimen. Such bending is due to the laterally uniform outward thrust of pins caused by the main member of the joint straining with respect to splices. This thrust is held in equilibrium by tension in the bolts. The bolts should be staggered or should be placed in two rows approximately at the quarter points in the width of the splice, in order to reduce this lateral bending to a minimum.

Joints of type A had 6 x 6-in. main



DISPLACEMENT OF MEMBERS OF JOINTS C, D, E AND F

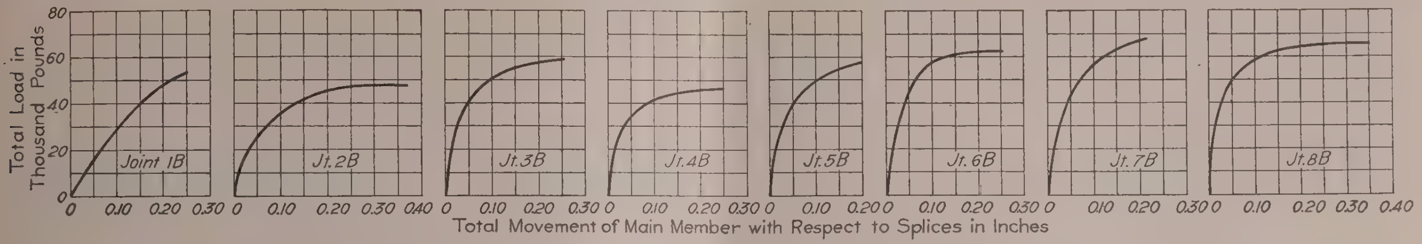


DIAGRAM SHOWING RELATIVE DISPLACEMENT OF MEMBERS OF JOINTS B UNDER TEST LOADS

members and 3 x 6-in. splices. They were bound with three 9/16-in. bolts, spaced 6 in. apart along the center lines of splices. There was a clearance of 3 in. between bolts and the ends of component parts of the joints. Cast-iron washers, ½ x 2½-in., were used throughout. Four 2-in. holes were drilled in the bolted joints, half way between bolt lines; into these the pins were driven. While in practice such joints are subjected to tension, for expediency in testing their resistance in compression was determined. The test results for these joints are given in Table 1.

JOINTS B, C, D, E AND F

With the exception of the pins, joints of type B were identical in design with the A joints. The test results for these joints are given in Table 2.

Two joints, C and D, had 12 x 12-in. main members and 6 x 12-in. splices. They were held together by eight ¾-in. bolts, placed opposite, with 6-in. pitch in two rows 7½ in. apart. Six Hawaiian Ohia pins were driven, three on a side, snugly into holes drilled midway between bolts. All washers were ½ x 2½-in. cast iron.

Joint E had a 12 x 12-in. main member and 6 x 12-in. splices. Bolts 1 in. in diameter were used, spaced 11 in. on centers and staggered in two rows 4 in. apart. The pins were of hickory. These were placed on 6-in. centers and midway between bolts. The cast-iron washers were 5 in. in diameter.

Joint F was identical in design with joint E, except that it had 3¾-in. instead of 5-in. washers.

Table 3 gives the test results for joints C, D, E and F.

METHODS OF TESTING

All joints were tested in an Olsen 200,000-lb. universal testing machine by resting the bases of the splices squarely on the table of the machine and applying through a spherical head compression upon the projecting end of the main member. For all joints, except A, the movement of the main member with respect to splices was determined. This was measured on the run, to 0.01 in., by means of pencil graduations marked on cards which were tacked to the specimens on opposite sides of each joint line, like vernier scales. These deformations were read for constant increments of loading, which varied in amount for different specimens according to their size. The relation between observed deformation and corresponding load for each joint is shown in the deformation diagram.

TABLE 1.—RESISTANCE OF TIMBER SPLICE JOINTS A IN COMPRESSION

		Maximum load, thousands of pounds			
No.	Kind of pin	Total	Per sq. in. of main member	Per lin. in. of pin	Cause of failure
1	Standard weight gaspipe	49.8	1.38	2.08	Pipes crushed
2	Oak	52.6	1.46	2.19	Wood sheared
3	Standard weight gaspipe	45.6	1.27	1.90	Pipes crushed
4	Standard weight gaspipe	44.4	1.23	1.85	Pipes crushed
5	Oak	51.3	1.42	2.14	Wood sheared
6	Oak	52.8	1.46	2.20	Wood sheared

Specimens with oak pins average 12 per cent higher than those with standard-weight gaspipe.

TABLE 2.—RESISTANCE OF TIMBER SPLICE JOINTS B IN COMPRESSION

		Maximum load, thousands of pounds				
No.	Kind of pin	Total	Per sq. in. of main member	Per lin. in. of pin	Slip at max. load, in.	Cause of failure
1	Extra-heavy pipe	53.4	1.48	2.22	¼	Crushing at pins.
2	Standard-weight pipe	47.9	1.33	1.99	⅜	Pipes crushed.
3	Extra-heavy pipe	59.6	1.65	2.48	¼	Crushing at pins.
4	Standard-weight pipe	46.0	1.28	1.91	¼	Pipes crushed.
5	Solid iron	58.4	1.62	2.43	⅜	Crushing at pins.
6	Solid iron	61.9	1.72	2.58	¼	Crushing at pins.
7	Hawaiian Ohia	68.1	1.89	2.83	⅜	Crushing at pins.
8	Hawaiian Ohia	66.0	1.83	2.75	⅜	Crushing at pins; one pin sheared.

In nearly every case failure was accompanied by splitting of both splices along bolt line near top.

TABLE 3.—RESISTANCE OF TIMBER SPLICE JOINTS C, D, E AND F IN COMPRESSION

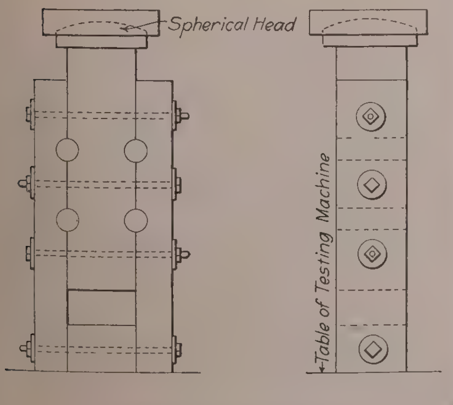
					Maximum load, thousands of pounds			
No.	Joint shown on	No. and kind of pins	No. and size of bolts	Size of washers	Total	Per sq. in. of main timber	Per lin. in. of pin	Slip at max. load, in.
1	Joint C	6 Hawaiian Ohia	8 ¾ in.	2½	158	1.10	2.20	9/16
2	Joint D	6 Hawaiian Ohia	8 ¾ in.	2½	163	1.13	2.26	½
3	Joint E	6 Hickory	4 1 in.	5	156	1.08	2.16	¾
4	Joint F	6 Hickory	4 1 in.	3¾	152.3	1.06	2.12	7/16

CONCLUSIONS

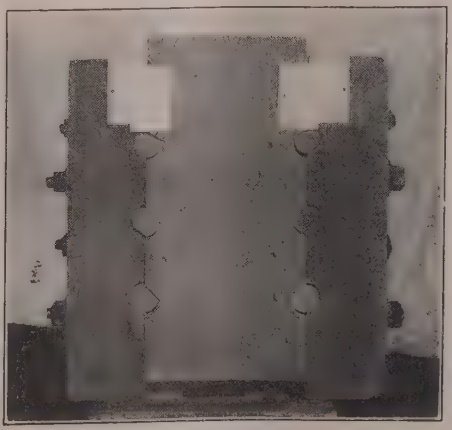
The following conclusions are drawn from the results in Tables 1, 2 and 3:

The efficiency of even the best types of timber joints is low. Assuming 6000 lb. per square inch as the ultimate resistance in compression of the grade of Douglas fir used in these tests, the efficiency of the various joints ranges between 18 and 31.5 per cent.

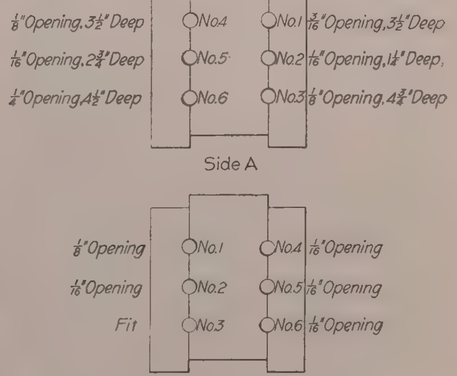
The efficiency of the larger joints is much less than that of the smaller ones. A comparison of the compressive stresses developed in main members of large and small joints in which Hawaiian Ohia pins were used shows that the large joints develop only about 60 per cent of the compression developed in the small ones. This undoubtedly is due to the difficulty in fabrication of a large joint so that it will have the same unit resistance that a smaller joint possesses, because the smaller joint is easier to make. The chief difficulty is to have close-fitting pins. If only a few pins fit snugly, then these must distort to an amount equal to the misfit of the others before the poorly fitting pins assume any of the load. Since strains below the elastic limit in structural materials are comparatively very small quantities, the necessity for perfectly fitting pins in such joints is very evident. The diagram of joint E indicates to what extent close-fitting pins may be expected in actual practice.



TYPICAL TEST SPECIMEN



TESTED JOINT E



JOINT-OPENING DIMENSIONS

Large joints deform under their maximum load slightly more than small joints. This may also be attributed to non-uniformity of pin bearing due to workmanship.

It is advisable to use a double line of bolts with large washers in order to reduce splitting of splices due to lateral bending. The splices of nearly all joints in Table 2, where there was a single center line of bolts, were split, while there was no splitting in the splices of joints in Table 3, where there were two lines of bolts and where large, strong washers were used in each case when the number of bolts was reduced.

If larger pins were used, the efficiency of the joints would be increased. Failure in all cases was due either to shearing or distortion of pins, or to failure of the Douglas fir under pin pressure. Increasing the size of pin would decrease the tendency toward any of these types of failure. There would be an increase in economy, however, only if the value of increase in efficiency would be greater than the added cost of larger pins plus the added cost of boring larger holes.

DEDUCTIONS FROM TABLE 3

Concerning the manner of failure of joints in Table 3 the following notes were recorded:

No. 1. Both splices were originally checked. Under the maximum load these checks opened wider. Four pins were cracked, including one which was sheared at one end.

No. 2. Three pins were cracked but not sheared.

No. 3. The workmanship on this joint was not good in that pins did not fit tightly. All pins were greatly distorted but did not shear.

No. 4. All pins were greatly distorted; one pin was sheared.

Failure in all four cases was due to crushing of Douglas fir due to pin pressure together with distortion of pins with or without pin shearing.

Railroad Contracting Methods Applied to Construction of Highways

Organization Problem and Transportation of Material at a Minimum Expense Features of Central Ohio Highway Work

RAILROAD contracting methods applied to a 24-mi. stretch of concrete-road construction in Central Ohio, between Zanesville and Hebron, promise to break all records for highway work. The organization problem and transportation of material at a minimum expense furnished two features of work. When the contract was awarded a railroad contractor found that he was the low bidder. As he had the railroad but not the highway equipment needed, he immediately associated with himself a firm of highway contractors having the necessary outfit. The work was then divided up by the highway contractor, the railroad contractor furnishing all material at a minimum cost and at a time saving which team haulage could not have approached. The detail plan of organization is shown herewith.

TRANSPORTATION FACILITIES

Owing to the fact that material could only be obtained for the work by haulage from either end, it was necessary to do away with teams and install a complete narrow-gage railroad system. To build this road 50,000 ties and 1500 tons of 60-lb. rail were laid. The rolling stock consists of six Vulcan engines and eighty-three cars of the Kilbourne & Jacobs and Western makes. Freight rates on material at the Hebron end were found to be higher than at the Zanesville end, and as a result it was found that money could be saved by hauling material 16 mi. from Zanesville, cutting the haul from Hebron down to 8 mi.

Four-car trains are run, each car carrying about 6 tons of material. The approximate cost per yard-mile, not including the cost of equipment, most of which the railroad contractor had on hand, is 13 cents, which compares with 28 cents estimated for team haulage. In fact, with 100 teams it was figured that the work would take about four times as long.

Foreign laborers are used and paid $1.75 per day. The cost of team hire is $5.50 per day. About 100 teams are employed for making short hauls in connection with the 130,000 cu. yd. of excavation necessary. On account of possible interruptions in delivery, sixty cars of stock (sand, coal and gravel) are kept on hand at each terminus. As the Baltimore & Ohio Railroad crosses the highway about 1½ miles west of Hebron it was necessary to terminate the narrow gage at this point. At the terminals a 20-ton Browning locomotive crane is used to unload material from the railroad cars to the storage piles and thence to a 16 x 36-ft. bin with three hoppers, from which gravel, stone and sand are delivered into the dump trains which run under the bins. At present the largest mileage of track runs from the Zanesville end, from which the longer haul will be made. One of the drawings shows the layout at the Hebron terminal, that at Zanesville being similar.

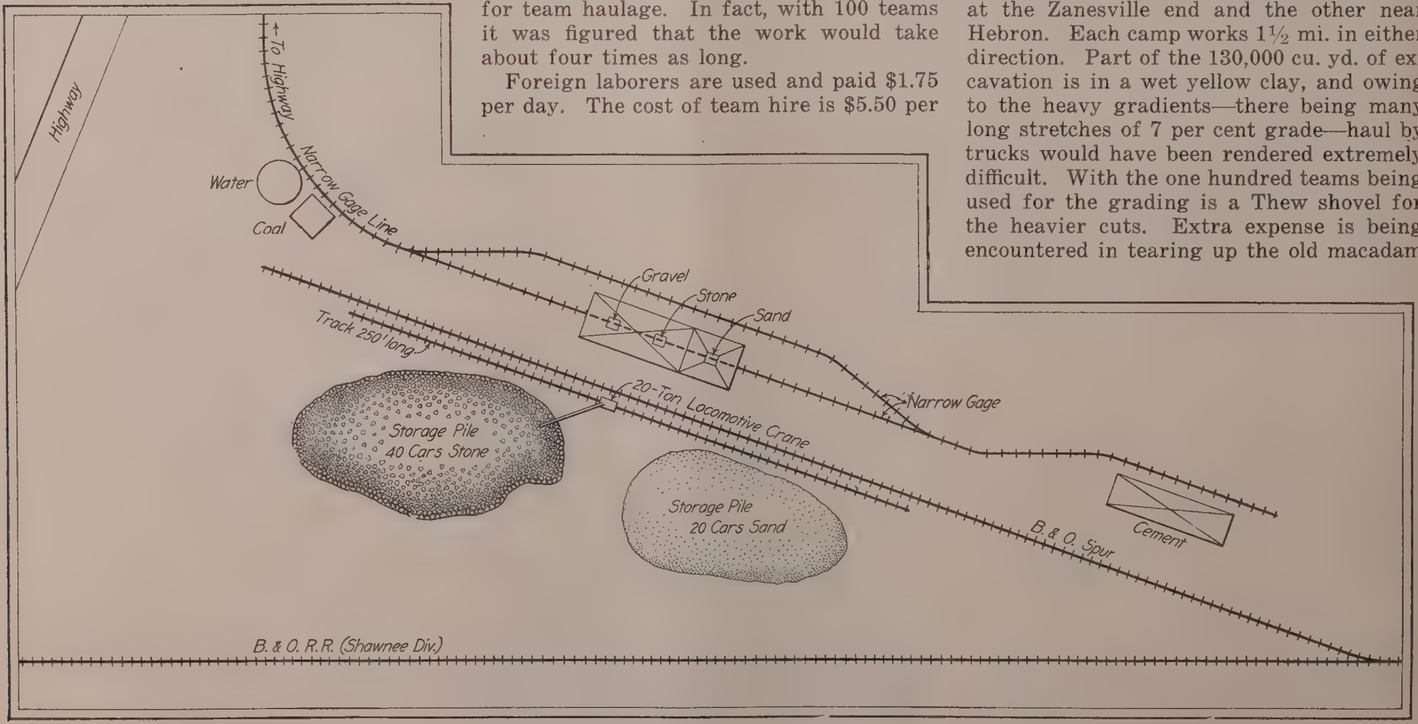
HIGHWAY CONSTRUCTION

The road itself is 6 ft. wide, with 9-ft. earth shoulders on each side, and will be built under joint specifications of the U. S. Office of Public Roads. A flat sub-base is obtained with a roller, on which a single course of concrete, 8 in. at the center and 6 in. at the side, is placed. A ⅛-in. tar-felt joint is placed every 30 ft. at an angle of 75 deg. with the center line.

Two Austin 15-ft. cube mixers are being used at present and two more will be started at work shortly. A 1:1½:3 mix is maintained at the Zanesville end, where a gravel aggregate is used. At the west end 1 part of Universal Portland cement is used with 1¾ parts of sand and 3 of crushed stone.

GRADING

Four grading camps have been established to date, each 3 mi. apart, three being at the Zanesville end and the other near Hebron. Each camp works 1½ mi. in either direction. Part of the 130,000 cu. yd. of excavation is in a wet yellow clay, and owing to the heavy gradients—there being many long stretches of 7 per cent grade—haul by trucks would have been rendered extremely difficult. With the one hundred teams being used for the grading is a Thew shovel for the heavier cuts. Extra expense is being encountered in tearing up the old macadam



LAYOUT OF HEBRON TERMINAL, SHOWING LOCOMOTIVE CRANE, AGGREGATE BINS, CEMENT SHED AND STORAGE PILES

HANDLING MATERIAL AT TERMINAL WITH LOCOMOTIVE CRANE AND HAULING IT WITH NARROW-GAGE TRAIN

pike, which, although it has had little improvement for many years, maintains a stubborn resistance to the plow.

To obtain water for keeping the road wet for ten days after setting, pumping stations have been established every 4 mi., each station pumping 2 mi. either way through 2-in. mains.

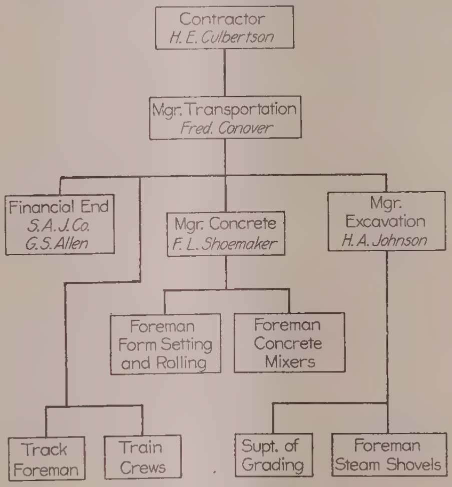
Method of Awarding Contract

The contract for the work was let April 15 to the H. E. Culbertson Company, of Cleveland, with which are associated Frank L. Shoemaker, H. A. Johnson and G. S. Allen, of the Globe Construction Company, all of Kalamazoo, Mich. R. N. Waid is the State resident engineer on the work.

The present contract was awarded in two sections, one covering the work in Licking County (10¾ mi.) and the other that in Muskingum County (13¾ mi.). The former county bonded the work itself for $140,000 and the latter for $100,000. The State highway department will furnish $80,000 and the Federal Government $120,000. The Federal appropriation was taken from a fund of $500,000, which was to have been distributed among the various States, but on account of the activity shown by its highway department, Ohio was enabled to secure the shares of a number of States which did not make any effort to secure Federal aid.

Mr. Culbertson, who has had a wide experience in railroad work, being at one time a division engineer on the Pennsylvania Railroad, is applying railroad contracting methods in the hope of finishing the work before cold weather sets in. The contract calls for completion by Dec. 15, 1914.

James R. Marker, State highway commissioner of Ohio, intends improving the entire National Pike from Wheeling to Columbus, a distance of 125 mi., which will call for an expenditure of about $2,000,000, the stretch from Zanesville to Hebron being the first contract to be awarded. Contracts for the entire mileage will probably be awarded during 1914. The National Pike is noted for its many heavy grades, but in rebuilding, these are, where feasible, being reduced or eliminated by detours. The cost per mile is an element of importance; owing to the Federal restrictions it could not exceed $16,000 to receive Federal aid.



ORGANIZATION DIAGRAM

Milwaukee's Lake Tonnage for 1913 exceeded every previous record, states the annual report of the harbor commission. The total movement was 8,875,231 tons, as against 7,772,965 tons in 1913 and 8,064,084 tons in 1910, the highest previous record.

Inclined Conveyor Belts Handling Wet Concrete

In completing the upper part of the Lake Spaulding dam, built in Central California for the Pacific Gas & Electric Company, it was necessary to elevate the concrete above the mixing plant in order that it could be conveyed several hundred feet along the crest of the structure. Instead of the usual hoist it was decided to install motor-driven, continuous belts for this purpose, and the 6-ply canvas belts with a ⅛-in. rubber coating which were used are reported to have given good satisfaction.

Beginning at the mixing plant, where four 1-yd. Smith mixers were employed, the concrete passed down a wooden chute which discharged onto a 28-in. belt moving at the rate of 420 ft. per minute and inclined upward at an angle of 20 deg. After being carried about 100 ft. on this belt, the concrete was discharged into a wooden flume, down which it flowed to discharge onto a second belt inclined upward at an angle of 18 deg. and operated at the same rate as the first. After this second lift the concrete could be chuted by gravity to the portions of the structure most remote from the mixing plant. Each belt was operated by a 20-hp, 220-volt motor.

About 240 cu. yd. per hour were moved. The liquid portions of the mix did not separate and run back on the 20-deg. slope, it is reported, and it was found possible to clear the belt of its load at the upper pulley. A revolving wire brush was first installed to clear the belt just as it returned over the upper pulley, but it was later found more effective to replace this with a compressed air blast. The air was directed against the belt at 100 lb. pressure through 1/32-in. holes 1 in. apart in a ½-in. pipe.

TWO ROAD-CONSTRUCTION VIEWS—STEAM SHOVEL WORKING IN HEAVY CUT AND GANG MIXING CONCRETE

Delay by Reason of Changes

Liquidated Damages—Extra Expense Caused by Changes and by Owner's Fault

By GEORGE A. KING
Of the Bar of the Court of Claims, Washington, D. C.

AN important decision has just been rendered by the Supreme Court of the United States, speaking through Mr. Justice Day, on the effect of delays on the part of the Government in carrying out its part of an engineering contract upon the liability of the contractor for liquidated damages for his delay in performance.

The case is that of the United Engineering & Contracting Company, which made a contract for a pumping plant for drydock 3 at the New York Navy Yard. The case was considered by the Court of Claims in an able opinion by Chief Justice Peelle and is more elaborate in its statement and reasoning than that just delivered by the Supreme Court. As that opinion is affirmed in its entirety by the Supreme Court, Chief Justice Peelle's opinion now has the authority of the Supreme Court in its favor.

Changes in Plans

The principal question arises out of a determination by the Government to make extensive changes in the floor of the pump well, followed by several supplemental contracts for the performance of the changed work. The specifications provided for liquidated damages at the rate of $25 a day for delay on the part of the contractor to complete in contract time. Up to the time of making these supplemental contracts no delay occurred on the part of the contractor. After the supplemental contracts were made, however, other delays occurred which were found by the Court of Claims to be chargeable to dilatoriness either on the part of the contractor or its subcontractor, which delayed completion beyond the time at which it might reasonably have been expected on the part of the contractor. The Government undertook to impose per-diem penalties of $25 for all the time of delay thus due to the fault of the contractor.

The Court of Claims, following a rule long since laid down by the English courts, held that a stipulated per-diem penalty being fixed by the contract to run from a certain date therein named for completion ran from that date, and that alone; and that if through an act of the Government that date was no longer applicable, as by extensions of time made by the Government for its own convenience, the per diem could only run from the date stipulated in the contract, and to make it run from a later date would be to make a new contract with the parties, and that therefore all right to charge liquidated damages was gone. It therefore held that the contractor was bound only to pay such actual damages as the Government could show it had sustained by reason of failure to complete within a reasonable time; and that no such damages having been shown, no deduction whatever was permissible.

Supreme Court Concurs

The Supreme Court, approving this view, says: "The precise question here is whether, when the work was delayed solely because of the Government's fault beyond the time fixed for its completion and afterward the work was completed without any definite time being fixed in which it was to be done, the claimant can be charged for the subsequent delays for which he was at fault by the rule of the original contract stipulating liquidated damages, or was that stipulation waived by the conduct of the Government and was it obligatory upon it in order to recover for the subsequent delays to show the actual damages sustained. We think the better rule is that when the contractor has agreed to do a piece of work within a given time and the parties have stipulated a fixed sum as liquidated damages not wholly disproportionate to the loss for each day's delay, in order to enforce such payment the other party must not prevent the performance of the contract within the stipulated time, and that where such is the case, and thereafter the work is completed, though delayed by the fault of the contractor, the rule of the original contract cannot be insisted upon, and liquidated damages measured thereby are waived. Under the original and first supplemental agreements the claimant knew definitely that he was required to complete the work by a fixed date. Presumably the claimant had made its arrangements for completion within the time named. Certainly the other contracting party ought not to be permitted to insist upon liquidated damages when it is responsible for the failure to complete by the stipulated date; to do this would permit it to recover damages for delay caused by its own conduct."

It has been a well-known practice in certain classes of construction for the Government, notably in drydock contracts, to make a contract with no very settled idea as to the full details of the construction and then to make radical changes by way of supplemental contracts. The decision of the Supreme Court is a wholesome reminder that after making these extensive changes there can be no ground for imposing damages upon the contractor for delay in completion for which the Government is thus damaged for its own convenience.

Additional Points

There are a number of additional points decided by the Court of Claims, small in amount in that case but of great importance in principle. As the action of the Court of Claims on all these points was approved by the Supreme Court, though in very few words, we refer to the decision of the Court of Claims giving the details of other rulings, one of which was as follows:

"There is no provision in the specifications or contract providing for gear casing, and as, under the custom then prevailing, such casing or covering was not supplied by the manufacturer, the claimant purchased and installed standard valves without such cases. The valves so installed were intended by the claimant to be opened and closed by a motor operated from a switchboard, so arranged by the claimant that workmen need not be in close proximity thereto. After the installation of these valves and while the work was progressing the defendants built a plank walk across the top of the pipes in the pumping plant for the more convenient use of its workmen, and then, to protect them from the high-speed gearing, the claimant incased the machinery at an extra expense to it of $479.05.

"The casing for protection was necessitated by the plank walk built by the Government, for which the claimant was in no way responsible, and the Government having received the benefit thereof the claimant is entitled to recover the amount so expended."

Another point of vast importance was that arising out of a breakage in a drain pipe as the result of sudden pressure caused by the Government in its water systems. It is thus stated in the opinion of the court:

"During the construction of the pumping plant in October, 1904, as set forth in Finding 11, there were in said navy yard two systems of water pipes, one a fresh-water system connected with the water system of the city of Brooklyn and the other a salt-water system belonging to the navy yard, with both of which systems of supply in the navy yard, as required by the contract, the claimant's system of priming the pumps in the pump well was connected. The pressure of these respective systems was subject to sudden increase for fire purposes in the salt-water system and to a cessation of use at times in the fresh-water system for city purposes.

Case of Broken Drain

"In the month of October, 1904, the volute of drain pipe 1, being a part of claimant's priming system, burst as a result of sudden pressure caused by the Government in one or the other of said water systems without notice to the claimant and without its fault. The claimant immediately repaired said break, and to forestall accidents of a like nature in future placed a reducing valve in said priming system. Notwithstanding this precaution, the elbow of drain pipe 2 was subsequently broken by like pressure in one or the other of said water systems. Claimant immediately repaired this break, the cost of all which to the claimant was $699.97.

"The Government contends that under paragraph 20 of the specifications the claimant is 'responsible for the entire work and every part thereof, and for all the tools, appliances, and property of every description used in connection therewith.' And further it is therein provided that the claimant 'shall specifically and distinctly assume all risks of damage or injury from any cause to property or persons . . . and to protect and defend the United States against all claims on account of any such damage or injury.'

"We do not think the construction contended for by the Government is well founded, as here the findings show that the breaks from the sudden pressure were caused by the Government without notice to the claimant and without its fault. The difficulties thus encountered were not such as claimant in the performance of its contract was bound to assume.

"The claimant had not agreed to assume the risks which the Government might impose by acts not contemplated when the contract was entered into, and for these reasons the claimant is entitled to recover on this item the amount so expended, $699.97."

These rulings are plainly but briefly approved by the Supreme Court in its decision and they therefore now have the sanction of the authority of the highest court.

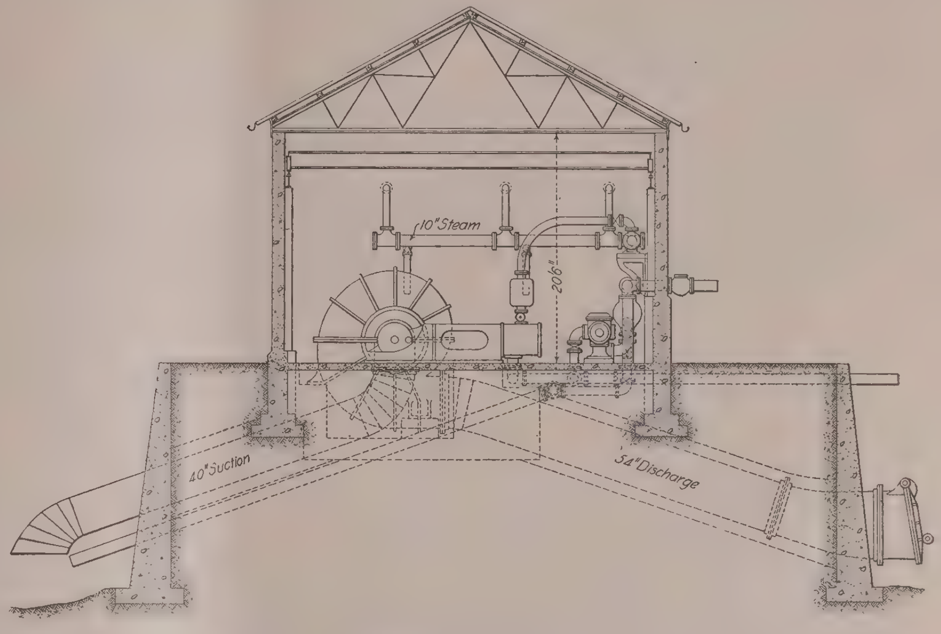
HIGHWAY IMPROVEMENTS in Chile during 1913 cost $500,000. This expenditure cared for the repair of 450 mi. of cart roads and the construction of 58 mi. of new highways.

Indiana Levee District

Thirty-one Miles of Levee and Two Pumping Stations with 250,000 Gallons per Minute Capacity

MORE than 31 mi. of levees, comprising about 1,235,000 cu. yd. of fill, are recommended for the William H. Brevoort et al. levee district, near Vincennes, Knox County, Indiana. Not less than eighteen public road crossings must be taken care of, requiring the placing of 18,510 cu. yd. of fill. Two pumping stations are proposed, one having a capacity of 150,000 gal. per minute and the other 100,000 gal. Sewers with flood gates will be constructed, buildings moved at four different points and breakwaters and bank protections provided.

The district is located south of Vincennes at the confluence of the Wabash and White Rivers. The levees will start at the intersection of the rivers and run 17 mi. up along the east bank of the Wabash River and 9 mi. along the north bank of the White River. In addition to these levees there are about 5 mi. of levees following ditches tributary to these rivers.

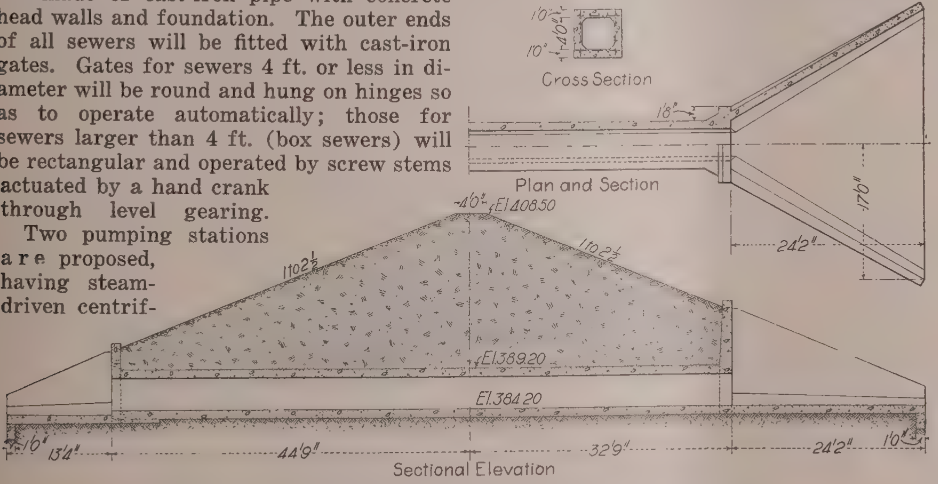


ELEVATION SHOWING GENERAL ARRANGEMENT OF PUMPING STATION

SEWERS

At different places sewers will be constructed through the levee. These sewers are divided into two classes—pipe sewers and reinforced-concrete box sewers. All 48-in. or larger sewers are built of reinforced concrete, and those less than 48 in. are made of cast-iron pipe with concrete head walls and foundation. The outer ends of all sewers will be fitted with cast-iron gates. Gates for sewers 4 ft. or less in diameter will be round and hung on hinges so as to operate automatically; those for sewers larger than 4 ft. (box sewers) will be rectangular and operated by screw stems actuated by a hand crank through level gearing.

Two pumping stations are proposed, having steam-driven centrifugal pumps. One plant will have a capacity of 150,000 gal. per minute and will be located near the point on the levee line across the Duchee River, about 15 mi. south of Vincennes. The smaller plant, of 100,000 gal. per minute capacity, will be located near Mantel Ditch, about 8 mi. south of Vincennes.



DETAILS OF TYPICAL BOX CULVERT THROUGH DRAINAGE LEVEE

At the pumping stations ditches will be excavated on both sides of the levee from the sewers. The inside ditch will serve as a suction pit and the outside one as the discharge pit for the pumps.

In station 1 there will be installed two pumps, each of a 75,000 gal. per minute capacity, designed to discharge against a head of 12 ft., including suction. The discharge pipes will be 54 in. in diameter and the pumps will have double-suction openings, each 40 in. in diameter. The pumps will operate at a speed of 125 r.p.m. when working against the maximum head. The accompanying drawing illustrates the installation. At the top of the pumps there will be a pipe connection for the priming by means of ejector or vacuum pump.

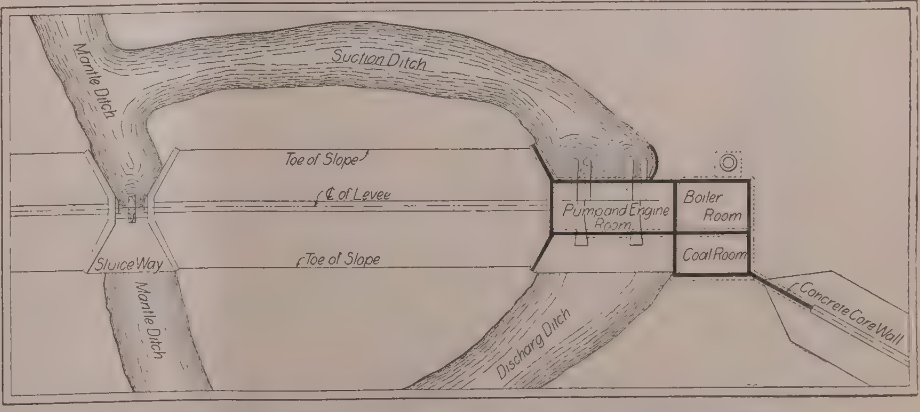
Three 78 in. x 18-ft. horizontal, return, tubular, steam boilers, each of 175 hp capacity and 150 lb. per square inch pressure, will be installed. The steam will be delivered to two horizontal, cross-compound, side-crank, throttling-governor engines directly connected to the centrifugal pumps. Each engine will develop 325 brake-hp when operating at a speed of 125 r.p.m. under 130-lb. pressure at the throttle valve.

Outside of the power house there will be erected a 120-ft. reinforced-concrete chimney 66 in. in diameter at the top. It will be designed to withstand a wind pressure of 50 lb. per square foot. Its lower 40 ft. will consist of a cylindrical double wall with an intervening air space.

Pumping station 2 will be similar to station 1, but as its capacity is smaller the pumps will have a capacity of 50,000 gal. per minute working against a head of 12 ft. The discharge pipes will be 48 in. in diameter and each of the double-suction pipes 36 in. The boiler battery will consist of two boilers of the same type and dimensions as those for station 1. The engines are also specified to be of the same type as those for station 1, but they will develop 216 brake-hp when operating at a speed of 150 r.p.m. with 130-lb. pressure at the throttle valve.

COSTS

It is estimated that the costs of establishing the levee district will be as follows: Wabash River levee, $104,416.42; Wabash River spur levee, $1,514.40; White River levee, $40,843.58; Plass Ditch levee, $8,211.65; River Duchee levee, $924.29; sewers, complete, $41,272.21; damages to C. & E. I. R. R. Company, $1,500; damages for moving buildings, $2,200; damages for right-of-way, $36,591; Spangler levee credits, $7,820; Plass Ditch levee credits, $6,096.46; Brevoort levee built credit, $15,453.36; pumping station 1, $80,000; pumping station 2, $62,000; muck ditches, ban-



LEVEE WITH PUMPING STATION AND SLUICEWAY WITH DITCHES quette embankment, sheet piling and bank protection, $22,489.63; engineering, attorneys, commission and courts, $14,590; contingent expenses on construction, $5,500—total estimate, $451,333.

The plans and specifications for this levee district have been prepared by J. S. Spiker, consulting civil and sanitary engineer, Vincennes, Ind. The petition to the commissioners asking for the plans and specifications was signed by William H. Brevoort, P. W. Lenahan and J. M. Spangler.

Experimental Concrete Road in Denver Park System

A MODEL mile of concrete highway is being built by the Denver Park Department in co-operation with two Portland cement companies as a means of illustrating the properties of that style of road. The road in question is a part of a 12-mi. stretch between Denver and Morrison, Colo., which is being improved as a part of the new mountain-park system. When the improvement was first contemplated, consideration was given to the construction of the entire distance of concrete, but the high cost and some uncertainty as to its durability made the department decide in favor of macadamizing the road for the present. However, the Colorado Portland Cement Company and the United States Portland Cement Company made a proposal, which was accepted, to build a mile of concrete road covering a stretch of adobe soil, the most difficult on which to maintain a macadamized road. The cement companies are constructing the road under expert supervision, while the park department did the necessary grading and prepared the road for laying the concrete. For the laying of the concrete and the work necessary to complete the road after grading was completed the park department pays the sum of $7000. Anything over this is paid by the cement companies. One-third of the road was completed by July 15.

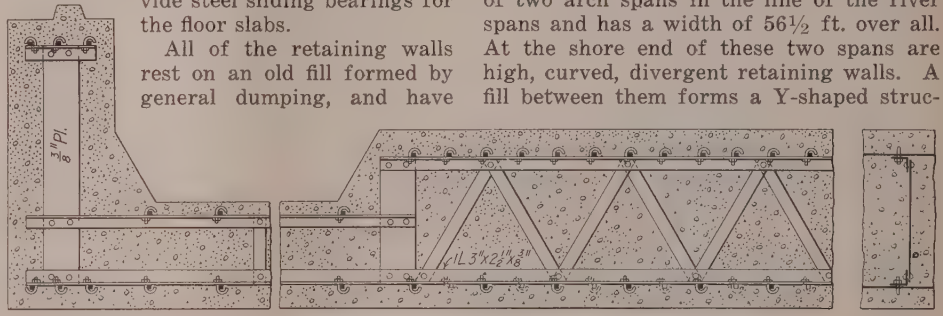
WATERWORKS MAINTENANCE training classes were established last winter at the St. Louis pipe yards for the purpose of providing instruction in various branches of pipe work to those employees who desired to avail themselves of the opportunity to learn. The classes were well attended and proved of mutual value to the men and the department.

Approaches to North Side Point Bridge over Allegheny River at Pittsburgh

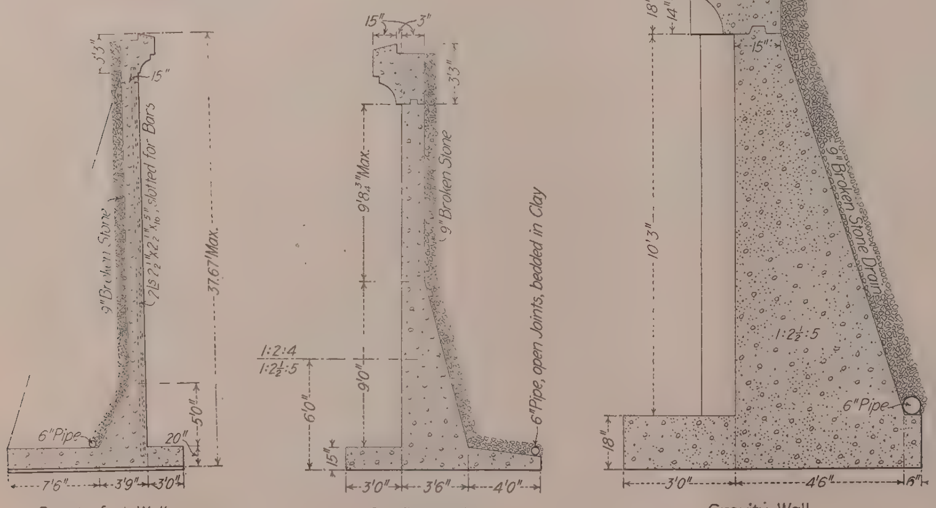
Consisting in Part of Reinforced-Concrete Arch Spans and in Part of Solid Earth-and-Slag Fill between High Retaining Walls

APPROACHES aggregating 1777 ft. in length will be required at the North Side Point Bridge across the Allegheny River near its confluence with the Monongahela at Pittsburgh. The two 531-ft. channel spans and their erection were described in the Engineering Record of July 18, page 72. The approaches consist of arch spans near the river, where the height is greatest and railroad tracks are to be crossed, and where embankments in the lower portions are made of slag with an earth core and retained by concrete walls of different types, varying with the height. These walls are provided with longitudinal expansion joints. The arch spans have transverse riveted steel members and provide steel sliding bearings for the floor slabs.

All of the retaining walls rest on an old fill formed by general dumping, and have extended footings at a considerable distance below the surface of the ground to provide sufficient bearing power to prevent settlement. These walls are provided with a longitudinal tongue-and-groove construction joint between the main body of the wall and the overhanging coping, designed to prevent cracking of the wall along this line due to shrinkage of the concrete. This cracking would presumably occur if the coping was concreted at the same time as the wall thereunder. The walls are constructed in sections, with vertical expansion joints which extend through the coping.



STRUCTURAL STEEL REINFORCEMENT STRUT IN ARCH SPAN



Counterfort Wall Cantilever Wall Gravity Wall

TYPICAL SECTIONS OF RETAINING WALLS UNDER DIFFERENT CONDITIONS

There are eight arch spans, varying in length from 40 ft. 4 in. to 85 ft. in the clear, with reinforced monolithic arch rings carrying vertical spandrel walls parallel to the axis of the bridge and vertical cross walls at right angles to the axis of the bridge, upon which rest the reinforced-concrete floor slabs. There is also a 20½-ft. flat-slab span over the undergrade crossing of a railroad track nearly at right angles to the bridge.

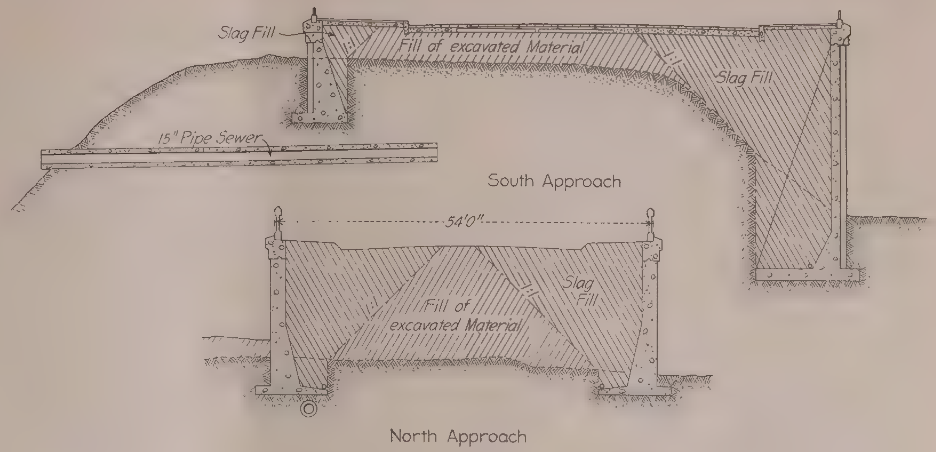
SOUTH APPROACH

The south approach, about 913 ft. long, attains an elevation of approximately 71½ ft. above pool-full elevation. The highest portion adjacent the river span is composed of two arch spans in the line of the river spans and has a width of 56½ ft. over all. At the shore end of these two spans are high, curved, divergent retaining walls. A fill between them forms a Y-shaped structure joining the other section of the approach to the adjacent Point Bridge over the Monongahela River. The remainder of the approach, approximately parallel with the Monongahela River, consists of a solid fill between parallel retaining walls 72 ft. apart over all. These walls are finished with heavy coping and paneled by exterior pilasters 41 ft. apart on centers.

The walls are designed to provide a vertical expansion joint at each pilaster. This part of the approach has a 48-ft. roadway and two 10¾-ft. sidewalks. The retaining walls are made lighter than would be required for earthfill, as it is proposed to use granulated blast-furnace slag for filling behind them, only using earth to form the central part of the fill. Granulated slag stands at a much steeper slope than earth, thereby greatly reducing the pressure on the retaining walls. Another reason for its use is that it settles far less than earthfill, thus involving much less in maintenance repairs. It weighs approximately 55 lb. per cubic foot and in this locality can be obtained as cheap as, or cheaper than, earthfill.

NORTH APPROACH

The north approach is 864 ft. long. About one-half of this approach consists of retaining walls and fill; the other half consists of six arch spans from 40 to 85 ft. long between piers, their rise varying to harmonize with the height of the structure above the surface of the ground. All of the retaining walls have footings on the ground extended beyond both faces of the wall. The arch piers have unsymmetrical footings with wide offsets on both sides corresponding to the unbalanced arch thrusts, and are supported on clusters of concrete piles. To provide for drainage in both approaches catchbasins are placed along the curb lines; they discharge into longitudinal and transverse sewers placed between the retaining walls. These sewers are composed of 15-in. terra-cotta pipes, which, on steep grades and under walls, are embedded in a solid mass of concrete, 2 ft. 9 in. square.

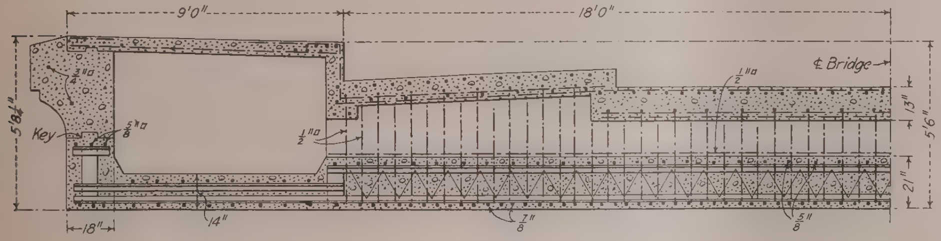


SECTIONS OF SOLID FILL APPROACHES, WITH FACING OF SLAG FILL

RETAINING WALLS

In general the highest retaining walls, up to a maximum of about 38 ft., are strengthened by counterforts 13½ ft. apart on centers. The lower walls, up to a maximum of about 15 ft. in height, are of a gravity type. Intermediate between the gravity and counterfort sections is the cantilever section, with a maximum height of about 23 ft. over all.

In all cases concrete which will be exposed to view in the finished structure is composed of a 1:2:4 mixture to allow for proper finishing. The rear of the wall is backed up with 9-in. broken stone to provide drainage to a 6-in. terra-cotta pipe, which is laid with open joints, in clay, on the rear of the footing, close to the battered surface and passing through the counterforts in the high walls.



HALF TRANSVERSE SECTION THROUGH ROADWAY AND CROWN OF ARCH

All arch spans have full arch rings with three-centered intrados and single-centered extrados, the axis of the curvature being at right angles to the center line of the bridge. The arch rings are constructed in monolithic strips between construction joints at the piers and abutments. There are also radial latticed struts perpendicular to the center line of the bridge, at the points where the transverse walls are supported on the arch rings. These struts are primarily used for attaching the reinforcing bars thereto and for properly spacing and holding the bars in position.

Near the crowns of the arches, where the heights of the transverse walls are comparatively small, the latter are cast integral with the arch rings, but where their heights are greater, near the ends of the spans, they are cast after the arch ring, and have horizontal seats with tongue-and-groove bearings to bond with the walls, which in all cases have full-length vertical reinforcement rods with their lower ends built into the anchor ring and their upper ends bent at right angles to anchor into the floor slabs.

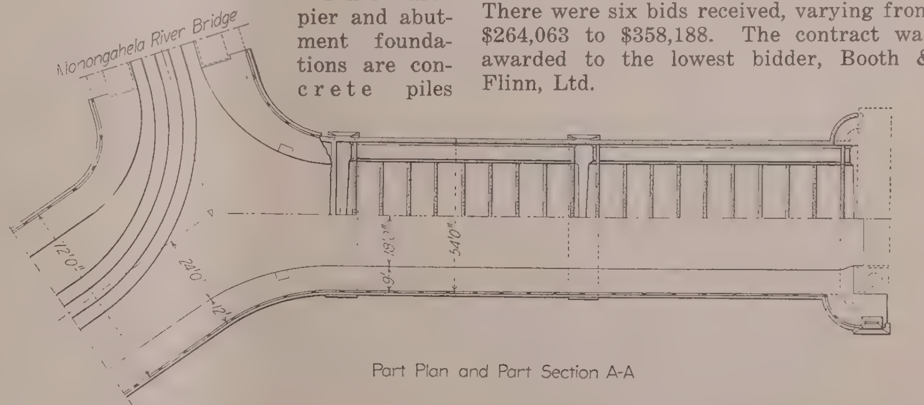
FLOORS OF ARCHES

The floor slabs are made continuous across the transverse walls, except at expansion joints, which are centered on the walls and have sliding bearings on the webs of channels placed on top of the transverse walls with their flanges down. The arch facing ring is cast integral with the arch ring proper, with a tongue-and-groove joint into the longitudinal spandrel walls. The upper part of the wall has a similar bond with a coping which forms an ornamental cornice, on which rest the sidewalk slabs and handrail posts.

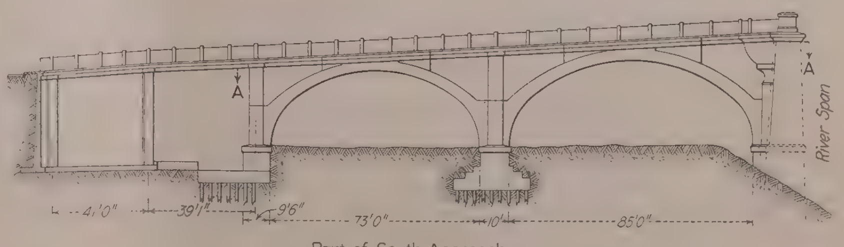
The transverse walls terminate at the curb lines, where they afford support for the longitudinal curb girders which carry the inner ends of the sidewalk slabs, thus leaving the space below the sidewalk unobstructed to accommodate water and gas pipes and electric conduits. Where the cross-walls are high, concrete struts extend from ends of the same to the spandrel walls. Just under the sidewalk, between the spandrel wall and the cross-walls, are placed I-beams, which are anchored at both ends, forming additional braces to the spandrel wall, and also providing supports for the water and gas pipes and electric conduits.

The arch pier and abutment foundations are concrete piles with reinforced-concrete pile caps supporting the pier shafts. The construction joints at the tops and bottom of the shafts are made with keys having one-half the area of the pier shaft. Horizontal joints, however, are not permitted on the tops of the piers, and the skewbacks are cast integral with the coping. The pile caps consist of three courses from 2 to 5 ft. in depth, with their ends offset unsymmetrically with the center line to correspond with the unbalanced thrusts of the adjacent arches. These caps are reinforced with horizontal rods at right angles with the axis of the pier, some of the rods being bent upward at the ends and some being looped around the pile tops to afford anchorage. The concrete foundation piles are spaced 3 ft. apart in each direction and are proportioned for 60-ton loads as determined by tests of at least one pile in each group. Raymond cast-in-place concrete piles were used. The average maximum settlement of ten piles tested was 3/16 in.

The structure was designed under the direction of the Bureau of Construction, Department of Public Works, of which N. S. Sprague is superintendent; Emil Swensson, consulting engineer; T. J. Wilkerson, division engineer in charge of the work, and John D. Stephenson, assistant engineer. There were six bids received, varying from $264,063 to $358,188. The contract was awarded to the lowest bidder, Booth & Flinn, Ltd.



Part Plan and Part Section A-A



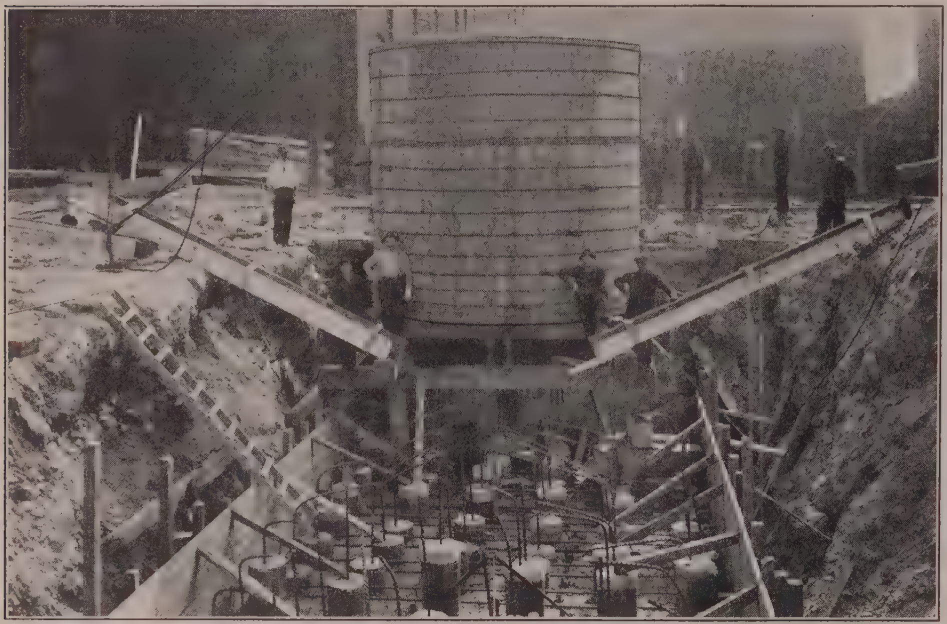
Part of South Approach

ARCH SPANS AND SOLID-FILL APPROACH WITH Y CONNECTION

Heavy Loading Tests of Concrete Piles

Cast-in-Place Piles Driven in Filled Ground and Subjected to 36-Hour 60-Ton Loads with 3/16-Inch Average Settlement

PROLONGED loading tests of eight concrete piles selected from a large number recently given for the foundations of the approaches to the North Side Point Bridge, Pittsburgh, showed very small settlement under static loads several times larger than are usually adopted for good wooden piles. The conditions and results of the tests and the special methods designed and executed to comply with the specification requirements, apply and remove the load easily and accurately and measure the settlement precisely are here described.



APPLYING 60-TON TEST LOAD TO CAST-IN-PLACE CONCRETE PILE

The bridge approaches, which are described on page 138 of this issue, are located on fills of slag and variable material overlying the sedimentary deposit of alluvial sand and loam on gravel bottom. The fluctuations of the river had compacted the overlying material to a considerable extent and the gravel was found to be so hard that it was impossible to secure deep penetration of the 1328 concrete foundation piles.

The piles are of standard construction 8 in. in diameter at the point, with a taper of 0.04 in. per foot, and have permanent reinforced steel shells driven by a No. 1 Vulcan steam hammer to a refusal of 1 in. for the last seven blows.

The specifications required that one pile in each group be selected for test by the director of the Bureau of Construction, Department of Public Works, Pittsburgh, of which N. S. Sprague is superintendent. The test required that after the pile was thirty days old a 30-ton load should be imposed and left undisturbed for 36 hr., after which the load must be increased to 60 tons and left undisturbed for 36 hr. more. Level readings on a steel dowel with a round top projecting from the pile were required to be taken, first, before the loading platform was placed on the pile; second, immediately after the 30-ton load was placed; third, 36 hr. after the 30-ton load was placed; fourth, when the 40-ton load was placed; fifth, when the 50-ton load was placed; sixth, when the 60-ton load was placed; seventh, 36 hr. after the 60-ton load was placed; eighth, when the load was reduced to 30 tons, and, ninth, immediately after the removal of the load and loading platform. To be acceptable it was required that the pile settlement should not exceed ¼ in. between the first and third readings, or ⅜ in. between the first and seventh readings, and must not vary more than 5/16 in. between the first and ninth readings. The bearing values of the groups in which the successful tests were made were based on one-half the loads causing ⅜ in. settlements in the test piles.

An axial 1-in. reinforcing bar projected several feet above the top of each pile, and in order to provide clearance for it a latticed steel column about 8 ft. long was seated on a steel cap plate grouted on the pile top and having a 2-in. hole through which the reinforcement bar projected in the center of the column. The dowel was omitted and readings were taken on a short graduated level rod set on the cap plate.

An I-beam and timber platform were balanced on the column and supported a tank filled with water to give the exact required loading. Level readings were taken by an instrument located 20 ft. away from the column, and the average maximum settlement of eight piles was 3/16 in., one-half of that permitted in the specification.

The piles varied in length from 12 ft. 11 in. to 22 ft. 4 in., the average being 17 ft. 10 in., with an area of 50 sq. ft. of surface in the ground. The minimum surface was 32 ft. and the maximum 69 ft. The tests were started from twenty-eight to forty-two days after the piles were driven, and occupied from five to eight days each, thus giving abundant time for settlement. The maximum settlement under the 60-ton load was 0.023 ft. and was observed on the shortest pile, which had an area of 32 sq. ft. in the ground. This corresponds to a load of 3715 lb. per square foot or to a load of 10,084 lb. per linear foot. The minimum settlements of 0.002 ft. and 0.009 ft. for loads of 30 and 60 tons were observed on a pile 17 ft. 9 in. long subjected to a load of 2400 lb. per square foot of surface and 7143 lb. per linear foot.

Booth & Flinn, Ltd., of Pittsburgh, are the general contractors for the approaches, and the piles were driven by the Raymond Concrete Pile Company.

Building a Sea-Level Road Between Rincon and Ventura, Cal.

CONSTRUCTION of the Rincon causeway, part of a sea-level road, reduces the distance between Ventura and Rincon, Cal., by 9 mi. For three stretches aggregating 6862 ft. the rocky shore line rises abruptly, but there is sufficient beach, covered more or less with water, so that a causeway seemed to the engineers the most feasible and economical way of carrying the road around the hill. Green eucalyptus piles with 10-in. butts and 7-in. joints cut from a nearby grove were driven not less than 8 ft. into the sand or clay beach. Where rock outcrops occurred the piles were set in holes 6 ft. deep and concreted in. There are 1200 piles in the work.

On top of bents of three piles each, which were capped out 9 ft. above high water, were placed the Oregon pine 12 x 12-in. caps, 24 ft. long, secured to the piles by ⅞ x 20-in. drift bolts. The bents are 15½ ft. on centers. Resting on the 4 x 12-in. stringers drift-bolted to the caps is the 3 x 12-in. decking, 24 ft. long, outside of which a rail 3 ft. high was erected.

John M. Williamson, of Santa Barbara, was contractor for the causeway.

Book Reviews

The report for 1913 on the surface-water supply of New Mexico has appeared in printed form. The book contains 216 pages and several diagrams and photographs. It is bound in brown cloth and its size is 6 x 9½ in. Its author is James A. French, State engineer. In addition to the report proper there are data on various floods in the State in 1913, data on various seepage and evaporation investigations and miscellaneous measurements and a list of the gaging stations maintained in 1913.

"Trollhättan, Its Canal and Hydroelectric Station" is the title of a series of books issued by the Royal Swedish Waterfalls Commission. Part 3 of the series has recently appeared, dealing with the power station. The book (which is written in Swedish) has three subdivisions—the first dealing with the hydraulic and other structures and the hydraulic equipment, the second describing the electrical equipment and appurtenant structures and the third treating of the transmission system and its substations. The first part is written by G. Malm, captain, Swedish Royal Corps of Engineers, who was in charge of the construction of the hydraulic parts of the plant. The volume, which is 10 in. wide and 12¾ in. high, contains 149 pages, 110 illustrations and two folded inserts. The two other parts are combined in one volume containing 186 pages, 91 illustrations and 30 inserts. The author is Einar Cronvall. He uses a series of lectures given by Torsten F:son Holmgren, who during the construction period was in charge of the electrical parts of the station. (Stockholm, Sweden, Kungliga Vattenfallsstyrelsen. 5 kroner per volume.)

Gate structures for irrigation canals are treated in Bulletin 115 of the U. S. Department of Agriculture. The bulletin contains 61 pages and numerous plates and figures illustrating structures that have proved practicable. Since it is prepared primarily for engineers and others familiar with gates and gate structures it does not attempt to treat the subject fully, but merely gives examples of installations that serve the purpose for which they are placed better than many other types in common use. Small and medium-sized structures are, for the most part, described, as it is believed that most of the problems confronting the engineer located in isolated regions of the West relate to this class. Local conditions control many features of design, and the descriptions given relate to existing structures in actual use. It is believed that these will prove suggestive and can readily be adapted to other conditions. One of the important questions dealt with is that of the materials to be used. The best practice, according to the department's investigator, seems to be to make a structure of combined wood and concrete, using concrete for the inaccessible parts. Local conditions affecting relative prices, states the investigator, will also help to determine the material to be used. (Washington, D. C., U. S. Department of Agriculture.)

Heat. By E. M. Shealy, Assistant Professor of Steam Engineering, University of Wisconsin. Cloth, 6 x 9½ in.; 262 pages; 110 illustrations. New York, McGraw-Hill Book Company, Inc. $2 net.

This book is written to supply the fundamental knowledge necessary for the understanding of heat-using machinery, such as the steam engine, the gas engine, the refrigerating machine and the air compressor. The book is mainly prepared for students in the extension division of technical universities, and its first part deals with fundamental laws relating to the nature, generation, transfer and transformation of heat. The latter part deals with machinery built along the principle of using heat in one form or another as motive power.

Engineering Geology. By Heinrich Ries, Ph.D., Professor of Economic Geology at Cornell University, and Thomas L. Watson, Ph.D., Professor of Economic Geology at the University of Virginia and State Geologist of Virginia. Cloth, 6 x 9 in.; 672 pages; 225 illustrations; 104 plates. New York, John Wiley & Sons, Inc. $4 net.

(Reviewed by Charles P. Berkey, Professor of Geology and Mineralogy, Columbia University, New York City.)

This is a book that should be welcomed by all engineers and students of engineering whose work brings them in close touch with geologic conditions. There has been no wholly acceptable textbook for use where the emphasis has to be kept on the applied side of the subject. Without raising the question of the relative values of pure versus applied science, one may as well recognize the fact that geology has actually become in one of its important phases, and to very many people, an applied science. The geologist has not always taken much interest in showing how a practical understanding of the principles of his science may be used by men chiefly engaged in other fields of work. But in these days there are many geologists whose principal investigations have to do with practical problems or with the discovery and elucidation of factors directly called for by engineers. On the other hand, there is a rapidly growing appreciation by engineers of the importance of the geologic factors in many undertakings where formerly such things were regarded as part of the unavoidable risks of the work.

This book is written by men who have a keen appreciation of this new viewpoint and who at the same time have a thorough knowledge of the science and a large experience both in teaching and in practical work. It departs far from the usual treatment of text books on geology. The historical, stratigraphic and paleontologic phases of the subject are left out entirely, except for occasional incidental reference, and the physiographic side is not given separate treatment. This gives opportunity for extended elucidation of the structural, petrographic, economic and dynamic phases of the science and for the introduction of many illustrations and explanatory cases. Rock formations, physical conditions and changes, field relations and products of economic and practical utility are given special attention—in some cases receiving more extended treatment than in elaborate treatises on geology. Structural features and modifications, weathering and soils, the geology of waters, both surface and underground, landslides, coastal changes and glacial deposits, are all treated in a comprehensive manner and with many illustrations and suggestions of a practical nature.

There are many well-chosen and well-arranged tabulations of values and aids to computation and results of tests that are usually found only in engineering books or special papers. On the whole, the treatment is calculated actually to make the connection between physical geology as a science and engineering needs.

The most striking departures from the usual text-book type are the inclusion of excellent chapters on minerals and rocks, on building stone, on limes, cements, clays and clay products, on coal, petroleum, natural gas, on road-building materials and on ore deposits. Some of these are of unusual importance to the engineer, and there is no doubt of the propriety of treating them in this thoroughly practical way in their normal relations or as a part of the general subject of geology.

There may be doubt of the advisability of entirely omitting the historical phase of the subject and a statement of physiographic principles as special points of view. A chapter on each would, in the opinion of the reviewer, add materially to the broadening influence of the book as a text for students and to the completeness of the book as a discussion of applied geology.

The book is suited to beginning classes of college grade or to the understanding of the average engineer or practical man, and it is at the same time advanced enough in statement of the principles involved to serve the uses of men of long training and experience. The reviewer recommends the book as worthy of serious consideration by engineers who have much to do with geologic matters and to applied science classes in geology.

Egyptian Irrigation. Vols. I. and II. By Sir William Willcocks, Formerly Adviser to the Ministry of Public Works, Constantinople, and J. I. Craig, Controller, Department of General Statistics, and Member of the Institut-Egyptien, Cairo, With an Introduction by Sir Hanbury Brown, Formerly Inspector General of Irrigation, Egypt. Cloth, 7½ x 10½ in.; 883 pages; 81 plates; 188 illustrations. New York, Spon & Chamberlain. $15.

This book now appears in its third edition. The first volume deals with the general conditions prevailing in the Nile Valley, its meteorology, the soil, analysis of the physical and chemical consistency of the Nile waters, the ground water and its movement, the evaporation and absorption, and an analysis of the crops, trees, animals, population, railways, roads, navigation and financial status of the country as a whole. Chapter 2 is focused on the Nile itself, giving its course, drainage area, slopes and velocities, discharges, etc., and the following chapter analyzes the river and its tributaries more in detail. Chapters 4 and 5 deal with basin irrigation, while Chapters 6 and 7 describe perennial irrigation in various parts of Egypt.

Volume II has eight chapters, dealing with drainage and land reclamation, the Nile in flood, engineering details, barrages, water storage and flood protection, the Assuan dam, agricultural matters and administrative and legal problems. The book is well written and its illustrations are well chosen. As the condition in the Nile is such as to make any work extremely difficult, this book will be of interest to engineers occupied in river improvement. The old rule that nature adjusts itself must be adhered to as closely as possible, and every river must be given time after one operation to adjust itself to the new conditions before other work is performed.

Foundations of Bridges and Buildings. By Henry S. Jacoby, Professor of Bridge Engineering, Cornell University, and Roland P. Davis, Professor of Structural and Hydraulic Engineering, West Virginia University. Cloth, 6 x 9 in.; 603 pages; 178 illustrations. New York, McGraw-Hill Book Company, Inc. $5 net.

(Reviewed by F. H. Constant, Professor of Civil Engineering, Princeton University, Princeton, N. J.)

The achievements of engineers in this most difficult field are recorded, for the most part, in the countless pages of the periodic journals. Chronicled in this isolated way they are generally narratives of methods adopted and difficulties overcome in individual and unrelated cases. The few books upon the subject generally retain the same narrative form without any serious attempt to relate the various methods to each other or to the nature of the problem.

It would perhaps be too strong a statement to assert that foundation methods are susceptible to strictly logical classification, so that given the conditions of the problem the proper method of solution may be determined with mathematical accuracy. One difficulty which will always exist is that the conditions cannot always, with any degree of certainty, be known beforehand. Yet there is what might be termed the science of foundation construction. Undoubtedly some methods are more adapted to certain conditions than others, and the experienced engineer is able to make the proper selection. The authors of this book have done more than simply to produce a valuable compilation of up-to-date foundation processes, generously illustrated by carefully selected examples of actual construction. They have made a distinct effort to present an orderly classification of various methods and to relate them to the particular problem. The engineering profession is indebted to them for the painstaking care and the great labor involved in the preparation of this work. It fills a long-felt want.

Although it was originally intended to confine the volume to 300 pages, it became necessary to expand it to twice that space in order to treat the topics satisfactorily. They deal almost entirely with American practice.

The book is wholly descriptive and is supplemented by frequent reference to other standard works in which the principles of theory and design are more specifically treated. As a rule, matter satisfactorily handled elsewhere has been excluded. Thus, although concrete forms the principal material in foundation construction, the usual chapters upon the properties of concrete, plain and reinforced, commonly found in the more recent books of this character, are not included.

It is not possible in a brief review to more than mention the topics covered in the subject. The first five chapters, including nearly 200 pages, cover the subject of piles. While, as was to be expected, concrete and metal-sheet piles receive very ample attention, timber piles are not slighted, but are handled in a refreshingly new and up-to-date manner.

The authors appear to incline to the use of the elaborate Goodrich formula in determining the bearing power of timber piles, but they attempt to reduce it to a simple working form by the introduction of constants and the elimination of unimportant terms. It is the old question, when so many unknown factors have to be assumed, whether the formula is as reliable as the engineer's practical judgment. A large set of readers will, however, be young engineers, who are generally aided by the use of a formula. Such a formula has the value, at least, of calling attention to the numerous elements which enter into the problem.

The nomenclature in common use in the engineering literature upon the subject of cofferdams, caissons and cribs is generally vague and confusing. The authors have here adopted a systematic classification which they hold to consistently and which goes far to clarify the subject. Chapters 6 and 7 cover cofferdams and caissons of the box and open types. Chapters 8 and 9 treat pneumatic caissons for bridges, and chapter 10 the same for buildings. Chapter 11 gives the open-well process in its various forms. The treatment of these important topics is full and modern and occupies about 200 pages.

Chapters 12, 13 and 14 cover, respectively, ordinary bridge piers, cylinders and pivot piers and bridge abutments, and illustrate the new types, including the hollow and arch forms, which have been introduced within the last decade. Spread foundations receive consideration in chapter 15 and include double column footings and reinforced-concrete wall and column footings. The important subject of the underpinning of buildings receives clear and systematic treatment in chapter 16, illustrated by many text cuts. The exploration and bearing power of the soil, which logically comes at the beginning of the work, is treated in chapter 17.

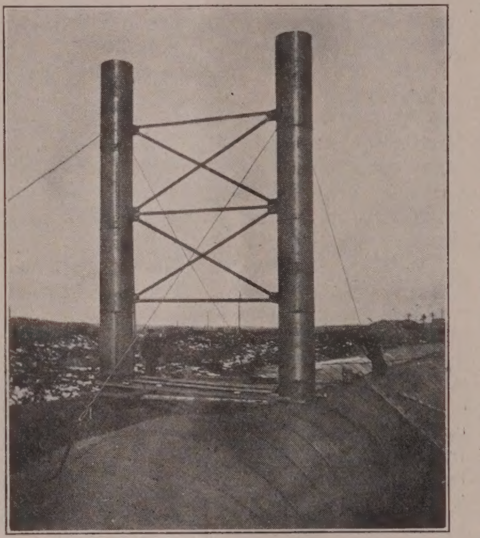
Chapter 18, which is on pneumatic-caisson practice, has been written by T. Kennard Thomson. It necessarily covers many things previously touched upon in the earlier chapters, but from the viewpoint of the experienced specialist. The last chapter contains 35 pages of selected and classified references to the engineering literature upon the topics covered in the text. This, especially for students who are acquiring the habit of reference reading, is one of the most valuable features of the book.

The style is clear and interesting. The text is abundantly illustrated with numerous drawings and some sixty full halftone photographs. The typographical work is uniformly excellent. If the book is intended primarily for the young practitioner it will prove equally valuable to the older engineer, who will find collected here in systematic form much matter which could only be found by laborious search in engineering periodicals or in the offices of engineers.

Letters to the Editor

Vents on Steel Pipe Lines

SIR: In the Engineering Record of May 23, page 594, there is an article entitled "Vents on Steel Pipe Lines." The general substance of the article contains very interesting information. I wish, however, to protest against the statements in the last paragraph: "The Engineering Record of July 22, 1912, page 92, has a description of the 15-ft. riveted-steel pipe line at Grand Falls, Newfoundland. A 36-in. riser pipe was placed at the summit to allow air to enter and to prevent collapse. The thickness of the pipe was not given, but assuming it to be 3/8 in. it will be seen from Table 1 that a vacuum of less than 1/2 lb. per square inch will cause the collapse of this pipe."


48-INCH VOMIT PIPES

In the first place, the 36-in. riser pipe referred to is in reality a 48-in. pipe. (The accompanying photograph shows the position and location of these pipes.) Second, the thickness of the plates, instead of being 3/8 in., is 7/16 in. Third, these pipes were not placed at the summit to allow air to enter and prevent collapse as stated in the article, but are what is known as "vomit pipes," and act in the capacity of a safety valve. If for any reason an excess internal pressure is set up, water in the penstocks will be backed up into, and may overflow at the top of, these pipes. The proper vent to allow air to enter and prevent collapse is taken care of at the forebay.

Such a paragraph may cause considerable trouble both to engineer and contractor, as it would appear from the article that the pipe line was not properly designed. I do not believe that such a paragraph should be allowed to pass without comment, inasmuch as the authors could have verified their information had they chosen to do so. Upon investigation these pipe lines will be found to be properly designed in every particular, and their successful operation for the past six years should be sufficient proof.

The pipe line in question will compare favorably with any job in the country and for verification of the above we would refer you to George F. Hardy, of New York City.

W. J. WALSH,
President, Walsh's Holyoke Steam Boiler Works.
Holyoke, Mass.

[At this late date it is impossible for the Engineering Record to check up the source of information from which the article on the Grand Falls development was written, although in justice to the authors Mr. Walsh criticises it must be admitted that their comments accord with the description to which they refer. A typographical error has been discovered in that the Grand Falls article was in the issue of July 22, 1911, and not that of July 22, 1912.—EDITOR.]

Break in Hatchtown Dam

SIR: Several reasons prompt me to comment on the published statement of Washington Jenkins, city engineer of Ogden, Utah, relative to the failure of the Hatchtown dam, appearing in the Current News of your issue of July 11, page 15. I was familiar with the construction of this dam from its inception and with its behavior since completion, so that I feel qualified to speak on the subject. The West has suffered a great deal in recent years through failures of engineering structures in connection with irrigation projects. An additional suffering has been imposed by the publication of unfair or exaggerated reports concerning them, all of which have had a tendency to put American irrigation practice in a bad light.

Constructive criticism is always welcome and valuable, and where poor engineering or outright negligence has been the cause of failures such criticisms can only have a healthy effect. Mr. Jenkins' account, however, contains a number of inaccuracies for which there is no excuse, as the facts in the case could easily have been ascertained. Therefore, although a correct description of the dam was published in the Engineering Record of June 27, page 726, the writer wishes to call attention to the inaccuracies contained in Mr. Jenkins' account.

He states that the dam was built without any core wall or cutoff wall of any kind. The facts are that tests were made to determine the character beneath the surface and that a timber sheet-piling cutoff was driven across the entire bed of the stream and well up into the natural slopes forming the abutments. There has been no intimation that the dam failed through faulty foundation. I fail to appreciate the intimated fault contained in the paragraph referring to the downstream slope being unprotected. Unless rock is available, cheaper than earth, it is not customary to use it in the lower part of earth-fill dams.

Instead of a concrete culvert serving as an outlet, this was a masonry culvert. The large springs referred to on the west end of the dam were in reality what might be termed seeps, the total flow being probably 1 sec.-ft. The statement is made that these were piped so as to lead into the outlet culvert. The fact is that the springs were collected by a system of tile drains and were led directly out along the natural bank to the lower toe of the dam. There never were any springs on the east end or abutment of the dam. This was a perfectly dry hill, composed of clay, gravel and sand intermingled. Mr. Jenkins may refer to the seepage which developed after the reservoir was first filled. Mr. Jenson's report published in the issue of June 27 covers these facts thoroughly.

Mr. Jenkins' conclusion that the dam was both poorly designed and poorly constructed might have had some value if backed by definite and correct statements and had pointed out where remedies should have been made. As it is it can serve no good purpose and only tends to throw everyone concerned with the dam into disrepute. I believe I am safe in saying that Mr. Jenkins never saw the dam during or after construction until after its failure, and reports as to faulty construction were doubtless obtained from "prominent citizens" whose statements were born of malice.

H. S. KLEINSCHMIDT,
Of Brown & Kleinschmidt, Engineers and Contractors.
Salt Lake City.

How the Ancients Would Have Controlled the Mississippi

SIR: Referring to the abstract in the Engineering Record of June 27, 1914, page 723, of a paper by Sir William Willcocks on the control of the Mississippi, I would like to point out that there is another work going on which is contributing a share to the extreme high water, and also extreme low water, in the Mississippi River. This is especially true in its upper waters, and, of course, what affects the upper parts affects the lower parts to some extent.

In Minnesota, Wisconsin, northern Iowa and the Dakotas the land is covered to a considerable extent with sloughs, ponds and small lakes which act as reservoirs, retarding the flow of water, so that in their natural state after a rainfall it would require, in some cases, weeks for the water to reach an open stream. These little reservoirs equalize the flow in the streams below.

The soil in these low places is very rich, consequently very valuable to the farmers. During recent years, therefore, large numbers of drainage ditches have been constructed to drain these sloughs and ponds. As a result the water flows off immediately and very much larger quantities of water reach the streams in a short time after rainfall, causing higher water than under natural conditions. Later when the water is well drained off the streams below have much lower water.

During the years 1908 and 1909, when with the bridge and building department of the Chicago, Milwaukee & St. Paul Railway, with headquarters in Indianapolis, I had occasion to see a few of the ditches being constructed. Even at that time they were having an effect on the Mississippi and Minnesota Rivers.

C. E. HENDERSON,
Assistant City Engineer.
Port Arthur, Ont.

Frost Action on Concrete Posts

SIR: I was greatly interested in G. C. Stone's letter and photograph in your issue of June 27, page 744, on "Frost Action on Concrete Posts." As Mr. Stone says that the lesson to be drawn from the facts he has so carefully stated is obvious to him, I think it a pity he did not choose to explain what the lesson is, for his letter, as it stands, is rather mystifying.

It would appear that Mr. Stone blames the lack of spiral or hooping reinforcement for the disintegration of the posts. I will not dispute the value of spiral or hooping reinforcement for bearing direct and temperature stresses, but I am well satisfied that neither has any waterproofing qualities, and I am equally well satisfied, from Mr. Stone's description, that the posts failed because the concrete was not waterproof.

The real cause of the failure probably is that the concrete must have been very dry when used, and of poor quality. The fact that there are rust streaks parallel to and opposite the reinforcing rods along the entire length of the posts proves the porosity of the concrete. The rusting of the rods at the top of the posts is due to rain water which soaked directly into the concrete, or to the water from the flooded playground which found its way up by capillary action. Either case indicates a very porous and, probably, a poor concrete.

Spiral or hooping reinforcement might have prevented the core of the posts from disintegrating by resisting the expanding force of the water during the process of freezing. The chances of such a thing taking place are very small indeed, but such reinforcement could not possibly have prevented the outer shell from falling to pieces. Not only would it have been useless in this case in preventing disintegration, but it would also have been equally useless in resisting the stresses to which these posts have been subjected, for, seemingly, they are only under a bending moment due to the wire netting, in which case the spiral would take up diagonal tension.

When concrete disintegrates through the action of water or any chemical action, the less reinforcement the better, for the steel rather helps the disintegration than retards it.

In my opinion the lesson to be derived from these posts is that concrete which is to be exposed to the action of water and frost ought to be made as waterproof as possible, that it should be poured rather wet and thoroughly rammed, and that it should have a liberal proportion of good cement.

M. J. LORENTE,
Stone & Webster Engineering Corporation.
Cambridge, Mass.

Panama Canal Tolls

SIR: My father, who located the Panama and Honduras railroads, and who, in 1852, explored routes for an interoceanic canal via the River Atrato, estimated the cost of a canal, by any of the routes then considered (including Panama), at from $300,000,000 to $500,000,000.

From a letter by C. E. Grunsky, in the Engineering Record of July 11, page 57, it appears that the entire cost of construction of the Panama Canal is about $375,000,000, and that the annual expenses will be about $5,250,000 for operation and maintenance and $250,000 for the annual payment to Panama, making a total of $5,500,000. Mr. Grunsky quotes Colonel Goethals as believing that the "tolls should pay outstanding indebtedness, fixed charges and operating expenses" and Prof. E. R. Johnson as suggesting that the entire cost of the canal should be charged to shipping and repaid to the United States, within ten years, by the users of the canal. Mr. Grunsky dissents from these suggestions and remarks that the investment has been made for the benefit of commerce and for our own military advantage, and not for profit or with the intention of recovering the cost from users. He therefore urges that for the present the tolls should be only such as to provide for operation, repairs, depreciation and the annual payment to Panama.

Gentlemen who deal with such figures commonly neglect the trifling matter of dividing by 100,000,000 (the approximate population of the United States). Performing this not difficult feat, we have the following per-capita figures:

PER-CAPITA COST OF PANAMA CANAL

		Cost per annum
Interest on cost of construction, $3.75, at 6 per cent		$0.23½
Operation and maintenance	$0.05¼	
Payment to Panama	.00¼	
		.05½
Total		$0.29

Now if, as Mr. Grunsky suggests, we are going to swallow the camel of, say, 24 cents (on account of construction), why need we strain at the gnat of 5½ cents for operation and maintenance?

Mr. Grunsky fitly refers to our national distinction of "having made a large financial contribution for the benefit of the world's commerce," and rightly says: "The investment in the canal might well be treated the same as all other Government investments in improvements for general benefit"; thus:

"When jetties were constructed at the mouth of the Mississippi River for the improvement of the river entrance, or when the breakwater was constructed at San Pedro for the improvement of Los Angeles harbor, or when Ambrose Channel was constructed for access to the harbor of New York, there was no thought of a charge upon shipping to recover the cost of these works or interest on this cost. They are in service as a benefit to all alike who would use them."

And he might have added that when a city builds and maintains streets it wisely refrains from seeking to reimburse itself by charging tolls for their use, even though some of the users may be committing the crime of seeking to divert "business" from the United States to "foreign" countries.

In a communication to the "Polytechnic Review" of Aug. 19, 1876, my father said: "If such a canal is ever to be made it should be by the co-operation of the governments of the leading civilized nations of the earth; and that not with a view to any direct return in the shape of tolls * * * but to the indirect advantages which would accrue to them all from the opening of so important a 'world's highway'."

We have had the good fortune to build the canal unaided; and the foregoing per-capita statement indicates that we need not now pass the hat around for contributions.

Mr. Grunsky urges that the charges for the use of the canal "should be nothing at all, or only a small portion of the total cost of the canal." What a pity that this sentence did not end with the words "nothing at all," and thus second the fine suggestion of Dr. Henry S. Drinker, president of Lehigh University, that the canal be thrown open, toll-free, to the world! The retention of "only a small portion" suggests the classical example of Ananias and Sapphira, who kept back part of the price. It is as though I were to say to my guest: "My house is open to you, and the charge shall be nothing at all—or only a small contribution to cover the cook's wages."

Mr. Grunsky has made a handsome step

forward from that obsolescent epoch whose watchword was "Show me first your penny," to that newer epoch, close at hand and rapidly materializing, when both nations and individuals shall vie with one another in efforts for the common good; and it is regrettable that he consents to the retention of a fly in his otherwise admirable ointment—a nickel in what looks almost like 30 cents.

As one working-class American, I stand ready to sacrifice my 29 cents (or even 30 cents, if need be) per annum for the pleasure of seeing my country do the handsome thing handsomely; and I should be sorry to pose as going the 24 cents and drawing the line at the extra nickel.

Let the canal be toll-free. It will repay us a thousandfold.

JOHN C. TRAUTWINE, JR.,
Philadelphia. Civil Engineer.

Amount of SO_3 in Portland Cement

SIR: Prof. A. Baykoff, president of the commission of the International Association for Testing Materials for the study of the influence of the amount of SO_3 in Portland cement, has communicated with the undersigned, who represent the American Society for Testing Materials on the commission, in a letter which reads as follows:

"In sending you this communication I have the honor to ask you to present your ideas on a desirable organization of the work of this commission. On my part I would make the following proposition:

"As has been indicated in the report of the Association of German Portland Cement Manufacturers, various countries decide the question of the maximum amount of SO_3 in Portland cement in a different way; some make no difference between Portland cement for fresh water and for sea water, others prescribe different limits according as the cement is to be used in fresh water or sea water. The first prescribe the following maximum content:

Country	Per cent of SO_3
Germany	2.50
England	2.75
Austria	2.50
Canada	2.00
United States	1.75
Russia	1.75

"The following countries prescribe different amounts for fresh water and sea water:

	Per cent of SO_3	
Country	Fresh water	Sea water
France	3.0	1.5
Brazil	3.0	1.5
Japan	2.0	1.5
Argentine Republic	2.4	1.2

"Based upon these views, the Association of German Cement Manufacturers has asked the sixth congress of the International Association for Testing Materials in New York to examine the subject, and have adopted the following resolution:

"'The Congress recommends generally that the conditions for accepting Portland cement for all uses be fixed uniformly at a maximum amount of SO_3, that is to say, at 2.5 per cent.'

"Thus the question proposed for the consideration of our commission divides itself into two parts, which can be formulated as follows:

"First, is it necessary to fix two different normals for the maximum admissible amount of SO_3 in Portland cement according as the cement is designed for use in fresh water or in sea water?

"Second, what should be the admissible amount of SO_3 in Portland cement under conditions conforming to the decision of the first question? Will it be necessary to use one or two normal figures?

"As the solution of these questions can only be made after a sufficient quantity of exact data is available, it seems to me to be necessary, first of all, to collect a sufficient number of such data. With this object, and independent of the researches which produce them or which are going to produce them, it will be useful to collect and discuss the greatest possible number of researches and the work devoted to the question of the influence of SO_3 on Portland cement.

"In presenting to you these considerations I beg you to communicate to me as soon as possible your opinion on the subject of the work of our commission and of the direction which you would desire it to take.

"Will you accept, sir, the assurance of my most distinguished consideration."

It seems to us that this matter should be put before the producers and users of Portland cement in the United States so that such proposals in regard to the work to be done and any data that may be available may be sent us for transmission to the president of the commission. With this object in view we are asking you to bring it to the attention of such persons through the columns of your journal. Communications are requested from all who are interested in the matter.

JOHN B. LOBER,
CLIFFORD RICHARDSON.

An Engineer's Canoe

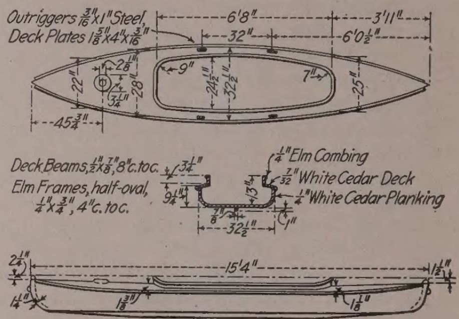
SIR: For certain kinds of reconnoitering work that engineers are often engaged in —such as going over a chain of small lakes to get data hastily relating to the probable water powers available—the little boat shown in the accompanying drawing has proved admirable. One or two men can sleep in it comfortably by putting their feet in the ends and their heads toward the center, using the sail for a boat tent, and mosquito netting or cloth at the ends. Thus nothing will get to them; even though they may roll a little in their sleep, snakes and other reptiles will not get nearer than the outside of the boat. This, as some who have been engaged on such work know from experience, is an important point.

I prefer to have a pair of oars, a good paddle and a small rectangular sail that can be rigged as a spritsail by reefing a corner back, with good, substantial grommets at the corners and about 1 ft. apart on all edges, and about 6 x 8 ft.; when used as a tent the 8-ft. way is put fore and aft of the boat, and very small rope passed through some of the grommet holes and under the boat enables the sail as a tent to be drawn tight against the boat.

A small box about 9 x 14 x 5 in. deep, outside measurements, is a great convenience and should be made with a hinged cover and lock. It forms a good seat for use in rowing and a convenient little place in which to lock up notebooks and small instruments.

One man can easily carry this boat, if well made of good light cedar, over a moderate portage, as it only weighs about 75 lb., or with the two oars, outriggers and rowlocks, a 5-ft. single paddle, stub mast and two spars with sail about 100 lb., and yet the boat is strong and stiff enough to stand hauling over logs and bars and stony places. If well built of first-class materials, with smooth skin outside and all joints made up with white lead and oil or waterproof marine glue, and all copper riveted with burrs, such a boat will cost with the fittings described from $75 to $100.

In having one built care should be taken that the bilge planking is made to a proper cup shape at the turn of the bilge, as otherwise it is likely sooner or later to check on the outside and become leaky. The advantage in having the bilge as sharp as shown is its readiness to stand right side up and not rock or roll easily—an important point in any boat, and particularly in one as small and light as this, in which a man often intrusts his life in strange waters.



DETAILS OF ENGINEER'S CANOE

I have traveled many miles in a boat like this with great comfort. I have been caught out on the Mississippi in summer squalls when neither bank could be seen and big steamboats have tied up, and have come through without danger. The boat rows easily and a man can make from 20 to 50 mi. a day under oars in still water.

Some points in the construction of the boat can perhaps be mentioned with advantage. One is the high combing, which somewhat stiffens the boat and deck, but is particularly advantageous in choppy water or when the boat is heavily loaded. Another point is the use of numerous small wide frames or ribs, which do not weigh much, but hold the planking together in a most satisfactory way, and being thin and near together, do not hurt an old man's bones when he lies on them with only a summer blanket under him.

F. W. SALMON,
Birmingham, Ala. Civil Engineer.

THE CEMENT GUN is being used to finish the ornamental concrete poles which are being erected on Van Ness Avenue in San Francisco. The poles are for the double purpose of lighting and carrying trolley wires for the new municipal railway. The coating is being applied after the poles are erected, so that the finish will be free from the marks of handling, and the pole proper will seem to be monolithic with the base. After the surface has been sufficiently coated it is immediately troweled to the desired lines, the aim being to secure a cement coating about 3/16 in. thick. The contract price for the finishing was $26 per pole.